VOL. L. OCTOBER, 1924. No. 8.

REPORTS, PAPERS, DISCUSSIONS, AND MEMOIRS

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REPORT OF THE
JOINT COMMITTEE ON STANDARD SPECIFICATIONS
FOR
CONCRETE AND REINFORCED CONCRETE

SUBMITTING

STANDARD SPECIFICATIONS FOR CONCRETE AND REINFORCED CONCRETE

AFFILIATED COMMITTEES

OF THE

American Society of Civil Engineers,
American Society for Testing Materials,
American Railway Engineering Association,
American Concrete Institute,
Portland Cement Association.

SUBMITTED TO CONSTITUENT ORGANIZATIONS,

AUGUST 14, 1924.

PREFACE

The Joint Committee on Standard Specifications for Concrete and Reinforced Concrete consists of five representatives from each of the following:

American Society of Civil Engineers,
American Society for Testing Materials,
American Railway Engineering Association,
American Concrete Institute,
Portland Cement Association.

This Committee is the successor of the Joint Committee on Concrete and Reinforced Concrete which was organized in Atlantic City, N. J., June 17, 1904, and was formed by the union of special committees appointed in 1903 and 1904 by the above-named organizations, except the American Concrete Institute which was added by invitation of the Joint Committee in 1915. The previous Committee presented progress reports in 1909 and 1912 and adopted a final report to its constituent organizations on July 1, 1916. It was the purpose of that Committee to prepare a Recommended Practice for Concrete and Reinforced Concrete. Its final report stated:

"The report is not a specification but may be used as a basis for specifications."

The present Joint Committee is charged with the preparation of Specifications for Concrete and Reinforced Concrete and in preparing these specifications is using as a basis the report of the former Joint Committee with such modifications as are necessary to make its recommendations agree with current practice, and such new data as mark advances in the art.

The initiative in bringing about the present Joint Committee was taken by the Committee on Reinforced Concrete of the American Society for Testing Materials on June 27, 1917, when the Committee voted to request the Executive Committee of the Society to invite the Member Societies of the previous Joint Committee to co-operate in the formation of a new Joint Committee. The Executive Committee approved this request on April 25, 1919, and an invitation was issued to each of the above-named organizations by the Executive Committee on behalf of the American Society for Testing Materials, to appoint five members on a Joint Committee on Specifications for Reinforced Concrete. The last of these organizations accepted the invitation on November 22, 1919. On January 21, 1920, a call for an organizing meeting on February 11, 1920, was sent by the Executive Committee of that Society to each of the twenty-five representatives of co-operating organizations, together with a list of members of the Joint Committee, and an outline of organization that had been previously submitted by the American Society for Testing Materials to and approved by the co-operating organizations.

The organizing meeting was held at the Engineers' Club, Philadelphia, Pa., and was called to order by George S. Webster, then Vice-President of the American Society for Testing Materials, who explained that he had been directed by the Executive Committee of that Society to act as Temporary

Chairman; he further stated that C. L. Warwick, Secretary-Treasurer of the Society, had been requested to act as Temporary Secretary until a formal organization of the Joint Committee had been effected.

The personnel of the Joint Committee is as follows:

American Society of Civil Engineers

W. A. SLATER, *Chairman,* Engineer-Physicist, Bureau of Standards, Washington, D. C.

MILTON H. FREEMAN,* Division Engineer, New York and New Jersey Bridge and Tunnel Commission, New York, N. Y.

A. E. LINDAU, President, American Wire Fence Company, Chicago, Ill.

FRANKLIN R. MCMILLAN,† Consulting Engineer, 628 Metropolitan Bank Building, Minneapolis, Minn.

SANFORD E. THOMPSON, Consulting Engineer, The Thompson and Lichtner Company, 136 Federal Street, Boston, Mass.

WILLIAM K. HATT,‡ Professor of Civil Engineering, Purdue University, Lafayette, Ind.

RUDOLPH P. MILLER,§ Consulting Engineer, New York, N. Y.

American Society for Testing Materials

RICHARD L. HUMPHREY, *Chairman,* Consulting Engineer, Philadelphia, Pa.

ALBERT T. GOLDBECK, Chief, Division of Tests, Bureau of Public Roads, Washington, D. C.

EDWARD E. HUGHES, Vice-President, Franklin Steel Works, Franklin, Pa.

HENRY H. QUIMBY, Consulting Engineer, Philadelphia, Pa.

LEON S. MOISSEIFF, Engineer of Design, Delaware River Bridge, Brooklyn, N. Y.

American Railway Engineering Association‖

J. J. YATES, *Chairman,* Bridge Engineer, Central Railroad of New Jersey, Jersey City, N. J.

T. L. D. HADWEN,¶ Engineer of Masonry Construction, Chicago, Milwaukee, and St. Paul Railway Company, Chicago, Ill.

FREDERICK E. SCHALL, Bridge Engineer, Lehigh Valley Railroad Company, Bethlehem, Pa.

C. C. WESTFALL, Engineer of Bridges, Illinois Central Railroad Company, Chicago, Ill.

GEORGE E. BOYD,** Formerly Division Engineer, Delaware, Lackawanna and Western Railroad Company, Buffalo, N. Y.

H. T. WELTY,†† Engineer of Structures, New York Central Railroad, New York, N. Y.

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* Appointed June 20, 1922, to fill vacancy.

† Appointed June 6, 1921, to fill vacancy.

‡ Resigned April 3, 1922.

§ Resigned April 25, 1921. Succeeded as Chairman by W. A. Slater.

‖ At the time of the completion of this report a vacancy existed in the representation of both the American Railway Engineering Association and the American Concrete Institute.

¶ Appointed November 21, 1921, to fill vacancy.

** Resigned November 7, 1921.

†† Resigned December 27, 1922.

*American Concrete Institute**

S. C. HOLLISTER, *Chairman,* Consulting Engineer, Philadelphia, Pa.

ROBERT W. LESLEY, Past-President, Portland Cement Association, Philadelphia, Pa.

ANGUS B. MACMILLAN,† Chief Engineer, Aberthaw Construction Company, Boston, Mass.

EGBERT J. MOORE, Vice-President, Turner Construction Company, New York, N. Y.

ARTHUR R. LORD,‡ President, Lord Engineering Company, Chicago, Ill.

LEONARD C. WASON,§ President, Aberthaw Construction Company, Boston, Mass.

Portland Cement Association

FREDERICK W. KELLEY, *Chairman,* President, Portland Cement Association, Albany, N. Y.

DUFF A. ABRAMS, Professor in Charge, Structural Materials Research Laboratory, Lewis Institute, Chicago, Ill.

ERNEST ASHTON, Chemical Engineer, Lehigh Portland Cement Company, Allentown, Pa.

EDWARD D. BOYER, Cement Expert, Atlas Portland Cement Company, New York, N. Y.

A. C. IRWIN, Engineer,‖ Structural Bureau, Portland Cement Association, Chicago, Ill.

J. E. FREEMAN,¶ Formerly Manager, Structural Bureau, Portland Cement Association, Chicago, Ill.

J. H. LIBBERTON,** Formerly Manager, Service Bureau, Universal Portland Cement Company, Chicago, Ill.

ORGANIZATION OF JOINT COMMITTEE

The Committee perfected a permanent organization on February 11, 1920, under the title "Joint Committee on Standard Specifications for Concrete and Reinforced Concrete" with the following officers:

Chairman, RICHARD L. HUMPHREY, Philadelphia, Pa.
Vice-Chairman, J. J. YATES, Jersey City, N. J.
Secretary-Treasurer, DUFF A. ABRAMS, Chicago, Ill.

and an Executive Committee consisting of these officers, and Rudolph P. Miller,†† New York, N. Y., and S. C. Hollister, Philadelphia, Pa.

* At the time of the completion of this report a vacancy existed in the representation of both the American Railway Engineering Association and the American Concrete Institute.

† Appointed October 19, 1920, to fill vacancy.

‡ Resigned April 17, 1923.

§ Resigned October 19, 1920.

‖ Appointed September 29, 1922, to fill vacancy.

¶ Appointed January 1, 1921, to fill vacancy; resigned September 14, 1922.

** Resigned December 31, 1920.

†† Succeeded by W. A. Slater, May 25, 1921.

The Committee adopted Rules of Organization and apportioned the work of preparing the specifications among sub-committees with the following personnel:

1.—*Materials (Other than Reinforcing):*
Albert T. Goldbeck, *Chairman,*
Duff A. Abrams,
Sanford E. Thompson,
J. J. Yates,
J. E. Freeman,*
J. H. Libberton.†

2.—*Metal Reinforcement:*
J. J. Yates, *Chairman,*
Duff A. Abrams,
Edward E. Hughes,
A. E. Lindau,
W. K. Hatt,‡
Arthur R. Lord.‖

3.—*Proportioning and Mixing:*
W. A. Slater, *Chairman,*
Duff A. Abrams,
Ernest Ashton,
T. L. D. Hadwen,
Henry H. Quimby,
George E. Boyd.§

4.—*Forms and Placing:*
T. L. D. Hadwen, *Chairman,*
Edward D. Boyer,
Angus B. MacMillan,
Egbert J. Moore,
Frederick E. Schall,
George E. Boyd,§ formerly *Chairman,*
Leonard C. Wason.¶

5.—*Design:*
S. C. Hollister, *Chairman,*
A. C. Irwin,
A. E. Lindau,
Franklin R. McMillan,
Egbert J. Moore,
W. A. Slater,
W. K. Hatt,‡
Arthur R. Lord,‖
H. T. Welty.**

6.—*Details of Construction and Fireproofing:*
Franklin R. McMillan, *Chairman,*
M. H. Freeman,
Leon S. Moisseiff,
C. C. Westfall,
Rudolph P. Miller, formerly *Chairman,*
W. K. Hatt,‡
Arthur R. Lord.‖

7.—*Water-Proofing and Protective Treatment:*
Frederick W. Kelley, *Chairman,*
Albert T. Goldbeck,
S. C. Hollister,
Robert W. Lesley,
C. C. Westfall.

8.—*Surface Finish:*
Henry H. Quimby, *Chairman,*
Edward D. Boyer,
A. C. Irwin,
Angus B. MacMillan,
J. E. Freeman,*
J. H. Libberton,†
Leonard C. Wason,¶
H. T. Welty.**

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* Resigned September 14, 1922.
† Succeeded by J. E. Freeman, January 1, 1921.
‡ Resigned April 3, 1922.
§ Succeeded by T. L. D. Hadwen, November 21, 1921.
‖ Resigned April 17, 1923.
¶ Succeeded by Angus B. MacMillan, October 19, 1920.
** Resigned December 27, 1922.

9.—*Form of Specifications:*

Richard L. Humphrey, *Chairman,*
Duff A. Abrams, *Secretary,*
T. L. D. Hadwen,
Albert T. Goldbeck,
S. C. Hollister,
Frederick W. Kelley,
Franklin R. McMillan,
Henry H. Quimby,
W. A. Slater,
J. J. Yates,
George E. Boyd,*
Rudolph P. Miller.†

MEETINGS

The Joint Committee held the following meetings:

1920:

Organization meeting, Philadelphia, Pa., February 11.
Second meeting, Asbury Park, N. J., June 23 and 24.
Third meeting, New York, N. Y., October 26, 27, and 28.
Fourth meeting, New York, N. Y., December 15, 16, and 17.

1921:

Fifth meeting, New York, N. Y., March 2, 3, and 4.
Sixth meeting, New York, N. Y., April 13, 14, and 15.
Seventh meeting, Asbury Park, N. J., June 21.
Eighth meeting, New York, N. Y., October 4 and 5.
Ninth meeting, New York, N. Y., December 9.

1922:

Tenth meeting, Cleveland, Ohio, February 17.
Eleventh meeting, Philadelphia, Pa., June 1 and 2.
Twelfth meeting, Philadelphia, Pa., September 19.
Thirteenth meeting, New York, N. Y., October 30 and 31.

1923:

Fourteenth meeting, Cincinnati, Ohio, January 25 and 26.
Fifteenth meeting, New York, N. Y., March 28.
Sixteenth meeting, New York, N. Y., June 6.
Seventeenth meeting, New York, N. Y., October 3, 4, and 5.
Eighteenth meeting, New York, N. Y., December 12, 13, and 14.

Numerous meetings of the Sub-Committees were held during the intervals between the meetings of the Joint Committee.

At those meetings the Joint Committee considered the reports of its Sub-Committees, which were edited by the Sub-Committee on Form and incorporated in the Standard Specifications for Concrete and Reinforced Concrete herewith submitted.

TENTATIVE REPORT SUBMITTED FOR CRITICISM AND DISCUSSION

The Joint Committee submitted to its constituent organizations Tentative Specifications for Concrete and Reinforced Concrete, June 4, 1921.‡ As pro-

* Succeeded by T. L. D. Hadwen, November 21, 1921.
† Succeeded by Franklin R. McMillan, June 6, 1921.
‡ *Proceedings,* Am. Soc. C. E., August, 1921, Papers and Discussions, p. 59.

vided in its Rules of Organization, these specifications have been open for criticism and discussion by the constituent organizations for a period of more than two years.

These specifications were discussed:

1.—At a two-day meeting of the American Society of Civil Engineers, New York, N. Y., covering six sessions on December 7 and 8, 1921, which discussion was continued for four months in the *Proceedings* of the Society.*

2.—At the Annual Convention of the American Concrete Institute, Cleveland, Ohio, February 15, 1922.

3.—The American Railway Engineering Association referred the specifications to its Committee on Masonry and the latter discussed them at several of its meetings.

4.—The American Society for Testing Materials referred the specifications to its several Committees that were particularly interested, and they were discussed at the Annual Meeting of the Society held at Atlantic Ciy, N. J., June 27-30, 1922.

5.—The Portland Cement Association referred the specifications to its Committee on Technical Problems.

Field Tests of Concrete

As a result of the discussion and criticisms the Joint Committee agreed to sponsor a series of field tests made under the auspices of a Joint Committee of Contractors, on which the Associated General Contractors of America was represented, with the object of determining whether the recommendations of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete were practical, especially as regards the provisions for the control of the quality of the concrete in the field.

Tests were made during the summer of 1923 on the concrete used in the construction of Building No. 10 of the Victor Talking Machine Company, Camden, N. J., through the co-operation of the owner and of Stone and Webster, Inc., the contractors; these tests have been completed and with a discussion of the results will be available in the near future.

A second series of tests was made of the concrete being used in the construction of the piers of the Newark Bay Bridge of the Central Railroad of New Jersey through the co-operation of the Railroad Company; these tests are practically completed; it is expected that other tests will be made during the present year.

All these tests have been made possible through the co-operation of the owners of the structures at which the tests were made, the U. S. Department of Commerce (Bureau of Standards), Department of Highways of the Commonwealth of Pennsylvania, the Joint Committee of Contractors, and the Structural Materials Research Laboratory, Lewis Institute, to whom the thanks and appreciation of the Joint Committee are extended.

The results† thus far obtained are a substantial justification of the Joint Committee's recommendations.

* *Proceedings*, Am. Soc. C. E., September, 1921, March, August, and November, 1922.

† For the Report on the Camden and Newark Bay Tests, see *Proceedings*, Am. Soc. C. E., November, 1924.

Another series of tests is to be inaugurated under the auspices of the Joint Committee on Concrete Reinforcement of the American Society for Testing Materials, having as an object the development of data as a basis for specifications for metal reinforcement.

Standard Specifications

The Joint Committee on Standard Specifications for Concrete and Reinforced Concrete has carefully reviewed the criticisms and discussions of its recommendations and the data that have become available since it submitted the Tentative Specifications in 1921, and has made such modifications in the specifications as were desirable.

The Committee is of the opinion that it should now submit the Standard Specifications for Concrete and Reinforced Concrete; with the later data as the result of tests now in progress it may be deemed desirable to submit revisions in the light of the information obtained.

The Committee in submitting the specifications directs attention to the fact that although the specifications are complete as regards the general uses of concrete and reinforced concrete, they may require some additional paragraphs to cover the use of these materials for special purposes.

The Committee has under consideration supplementary specifications covering special uses of concrete and reinforced concrete on which it is not prepared to report at this time.

The Joint Committee further calls attention to the fact that it has undertaken to prepare specifications covering the fundamentals to be observed in the general use of concrete and reinforced concrete; no attempt has been made to cover the details involved in the use of these materials in special structures. Although the sections relating to design deal primarily with building construction, nevertheless the principles involved are of general application to structures of other types.

This report has been submitted to letter ballot of the Joint Committee which consists of 23 members,* representing the five societies, 22 of whom voted affirmatively, 0 negatively, and 1 has refrained from voting.

Respectfully submitted,

Richard L. Humphrey, *Chairman.*

J. J. Yates, *Vice-Chairman.*
Ernest Ashton,
Edward D. Boyer,
Milton H. Freeman,
Albert T. Goldbeck,
T. L. D. Hadwen,
S. C. Hollister,
Edward E. Hughes,
A. C. Irwin,
Frederick W. Kelley,
Robert W. Lesley,
Duff A. Abrams, *Secretary-Treasurer.*
A. E. Lindau,
Angus B. MacMillan,
Franklin R. McMillan,
Leon S. Moisseiff,
Egbert J. Moore,
Henry H. Quimby,
Frederick E. Schall,
W. A. Slater,
Sanford E. Thompson,

* At the time of the completion of this report a vacancy existed in the representation of both the American Railway Engineering Association and the American Concrete Institute.

STANDARD SPECIFICATIONS
FOR
CONCRETE AND REINFORCED CONCRETE

SUBMITTED BY THE
JOINT COMMITTEE ON STANDARD SPECIFICATIONS
FOR CONCRETE AND REINFORCED CONCRETE

AFFILIATED COMMITTEES

OF THE

American Society of Civil Engineers
American Society for Testing Materials
American Railway Engineering Association
American Concrete Institute
Portland Cement Association

AUGUST 14, 1924

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* The Standard and Tentative Specifications and Methods of Tests are those of the American Society for Testing Materials, 1315 Spruce St., Philadelphia, Pa. See Book of A. S. T. M. Standards for 1924; also, Tentative Standards for 1924.

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STANDARD SPECIFICATIONS
FOR
CONCRETE AND REINFORCED CONCRETE

CHAPTER I
General Instructions

1.—These specifications cover the general conditions affecting the use of concrete and reinforced concrete. To use them in connection with the contract it will be necessary for the Engineer to:

(*a*) Provide the detail specifications covering the work in particular in which the concrete and reinforced concrete are to be used;

(*b*) Section 7, Table 1, insert percentages required;

(*c*) Section 10, fill in the blank indicating percentage of mortar strength required of fine aggregate as compared with standard Ottawa sand;

(*d*) Section 14, Table 2, insert sieve sizes and percentages required;

(*e*) Section 21, strike out one of the titles of the specifications for reinforcement bars; if billet steel bars are to be used, indicate whether structural, intermediate, or hard grade;

(*f*) Section 24, strike out one of the titles of the specifications for structural steel shapes;

(*g*) Section 28, strike out "volume" or "weight;"

(*h*) Section 29, if no tests of concrete are to be made, strike out the last paragraph in accordance with the footnote. In Table 4, insert the concrete strengths which are assumed as the basis for design of the different portions of the work;

(*i*) Section 30, Table 5, insert the slumps required;

(*j*) Section 51, strike out the method or methods for depositing concrete under water which are not applicable to the work;

(*k*) Chapter X, strike out sections on surface finish which do not apply;

(*l*) Section 95, strike out one of the two sections on terrazzo finish.

CHAPTER II
Definitions

2.—The following definitions give the meaning of certain terms as used in these specifications:

Acid Proofing.—Treatment of a concrete surface in order to provide resistance to the action of acids.

Aggregate.—Inert material which is mixed with Portland cement and water to produce concrete; in general, aggregate consists of sand, pebbles, gravel, crushed stone, or similar materials. (See *Fine Aggregate; Coarse Aggregate.*)

Anchorage.—The embedment in concrete of a portion of a reinforcement bar, either straight or with hooks, designed to prevent pulling out or slipping of the bar when subjected to stress. (The anchorage of tension reinforcement in beams includes only the embedded length beyond a point of contraflexure or of zero moment.)

Approved.—Meeting the approval of, or specifically authorized by, the Engineer.

Buttressed Retaining Wall.—A retaining wall with brackets or buttresses on the side opposite the pressure face uniting the upright section with the toe of the base.

Cantilever Retaining Wall.—A reinforced concrete wall having an upright section and a base each of which resists by cantilever action the pressure to which it is subjected.

Cellular Retaining Wall.—A retaining wall with a base, longitudinal upright sections, and a series of transverse walls, dividing the space between the longitudinal sections into cells which are filled with earth or other suitable material.

Coarse Aggregate.—Aggregate, subject to specified tolerances, retained on a No. 4 sieve and of a maximum size generally not larger than 3 in. (See *Aggregate; Fine Aggregate.*)

Column.—An upright compression member the length of which exceeds three times its least lateral dimension.

Column Capital.—An enlargement of the upper end of a reinforced concrete column designed and built to act as a unit with the column and flat slab.

Column Strip.—A portion of a panel of a flat slab which has a uniform width equal to one-fourth of the panel length perpendicular to the direction of the strip, and the outer edge of which lies on the edge of the panel. (See *Middle Strip;* also, Appendix I, Fig. 15.)

Composite Column.—A circumferentially reinforced concrete column with a core of structural steel or cast iron which is designed to carry a portion of the load.

Concrete.—A mixture of Portland cement, fine aggregate, coarse aggregate, and water. (See *Mortar.*)

Consistency.—A general term used to designate the relative plasticity of freshly mixed concrete or mortar.

Counterforted Retaining Wall.—A reinforced concrete wall with brackets or counterforts on the pressure face uniting the upright section to the heel of the base.

Crusher-Run Stone.—Unscreened crushed stone. (See *Stone Screenings.*)

Cyclopean Concrete.—Concrete in which stones weighing more than 100 lb. are individually embedded.

Dead Load.—Weight of the permanent parts of the structure.

Deformed Bar.—Reinforcement bar with shoulders, lugs, or projections formed integrally with the bar during rolling.

Diagonal Direction.—A direction parallel or approximately parallel to the diagonal of the panel of a flat slab.

Dropped Panel.—The structural portion of a flat slab which is thickened throughout an area surrounding the column capital.

Effective Area of Concrete.—The area of a section of the concrete which lies between the tension reinforcement and the compression surface in a beam or slab.

Effective Area of Reinforcement.—The area obtained by multiplying the right cross-sectional area of the metal reinforcement by the cosine of the angle

between its direction and that for which the effectiveness of the reinforcement is to be determined.

Engineer.—The engineer in responsible charge of the work.

Fine Aggregate.—Aggregate, subject to specified tolerances, passing through a No. 4 sieve. (See *Aggregate; Coarse Aggregate.*)

Flat Slab.—A concrete slab having reinforcement bars extending in two or more directions without beams or girders to carry the load to supporting members.

Footing.—A structural unit used to distribute wall or column loads to the foundation materials.

Gravel.—Rounded particles larger than sand resulting from the natural disintegration of rocks. (See *Sand.*)

Laitance.—Extremely fine material of little or no hardness which may collect on the surface of freshly deposited concrete or mortar, resulting from the use of excess mixing water, and usually recognized by its relatively light color.

Live Load.—Loads and forces other than the dead load.

Middle Strip.—The portion of a panel of a flat slab which extends in a direction parallel to a side of the panel, the width of which is one-half the panel length at right angles to the direction of the strip and the center line of which lies on the center line of the panel. (See *Column Strip;* also, Appendix I, Fig. 15.)

Mortar.—A mixture of Portland cement, fine aggregate, and water. (See *Concrete.*)

Negative Reinforcement.—Reinforcement so placed as to take tensile stress due to negative bending moment.

Oil-Proofing.—Treatment of a concrete surface for the purpose of preventing the penetration of, and resisting the action of, oils.

Panel Length.—The distance in either rectangular direction between centers of two columns of a panel.

Pedestal.—An upright compression member the height of which does not exceed three times its least lateral dimension.

Pedestal Footing.—A column footing projecting less than one-half its depth from the faces of the column on all sides and having a depth not more than three times its least width.

Plain Concrete.—Concrete without metal reinforcement.

Portland Cement.—The product obtained by finely pulverizing clinker produced by calcining to incipient fusion an intimate and properly proportioned mixture of argillaceous and calcareous materials, with no additions subsequent to calcination except water and calcined or uncalcined gypsum.

Positive Reinforcement.—Reinforcement so placed as to take tensile stress due to positive bending moment.

Principal Design Section.—The vertical sections in a flat slab on which the moments in the rectangular directions are critical. (See Section 143.)

Ratio of Reinforcement.—The ratio of the effective area of the reinforcement cut by a section of a beam or slab to the effective area of the concrete cut by that section.

Rectangular Direction.—A direction parallel to a side of the panel of a flat slab.

Reinforced Concrete.—Concrete in which metal is embedded in such a manner that the two materials act together in resisting forces.

Rubble Aggregate.—Stone or gravel larger than 3 in. in diameter and weighing not more than 100 lb.

Rubble Concrete.—Concrete in which pieces of rubble aggregate are individually embedded. (See *Rubble Aggregate.*)

Sand.—Small grains resulting from the natural disintegration of rocks. (See *Gravel.*)

Screen.—A metal plate with closely spaced circular perforations. (See *Sieve.*)

Sieve.—Woven wire cloth with square openings. (See *Screen.*)

Slump.—The shortening of a standard test mass of freshly mixed concrete, used as a measure of workability in accordance with the standard method. (See Appendix XI.)

Standard Sand.—Natural sand from Ottawa, Ill., screened to pass a No. 20 sieve and retained on a No. 30 sieve, used as the fine aggregate in standard strength tests of Portland cement. (For specifications, see Appendix II.)

Stone Screenings.—Crushed stone, subject to specified tolerances, passing through a No. 4 sieve. (See *Crusher-Run Stone.*)

Strut.—A compression member other than a column or pedestal.

Tremie.—A water-tight pipe of suitable dimensions, generally used in a vertical position, for depositing concrete under water.

Wall-Beam.—A reinforced concrete beam which extends from column to column along the outer edge of a wall panel.

CHAPTER III

Quality of Concrete

3.—*Strength.*—The design of the various portions of the structure is based on the assumption that the concrete will develop the compressive strengths at 28 days, which, for purposes of design only, are given in Section 29, Table 4.

4.—*Tests of Field Specimens.*—The making, curing, and testing of field concrete test specimens shall be carried out in accordance with the "Standard Methods of Making and Storing Specimens of Concrete in the Field" (Serial Designation: C31-21) of the American Society for Testing Materials (Appendix XIII).

CHAPTER IV

Materials*

A.—Portland Cement

5.—*Portland Cement.*—Portland cement shall conform to the "Standard Specifications and Tests for Portland Cement"† (Serial Designation: C9-21)

* These specifications cover the usual requirements for aggregate. Where concrete having special properties is essential the specifications should make clear the characteristics of the aggregate to be used.

† These specifications are also the standard of the following: United States Government, American Railway Engineering Association, American Concrete Institute, Portland Cement Association, and have been approved by the American Engineering Standards Committee as an American Standard.

of the American Society for Testing Materials (Appendix II), and subsequent revisions thereof.

B.—Fine Aggregate

6.—*General Requirements.*—Fine aggregate shall consist of sand, or other approved inert materials with similar characteristics, or a combination thereof, having clean, hard, strong, durable, uncoated grains and free from injurious amounts of dust, lumps, soft or flaky particles, shale, alkali, organic matter, loam, or other deleterious substances.

7.—*Grading.*—Fine aggregate shall range in size from fine to coarse within the limits indicated in Table 1.

TABLE 1.—Grading of Fine Aggregate.

Item.	Percentage by weight.
Passing through No. 4 sieve, not less than	(*a*)*
Passing through No. 50 sieve, not more than	(*b*)*
Passing through No. 50 sieve, not less than	(*c*)*
Weight removed by decantation, not more than	(*d*)*

* The grading requirements will vary with the type of work and characteristics of materials available in any given locality. Wherever practicable the following values should be inserted: (*a*) = 85%; (*b*) = 30%; (*c*) = 10%; (*d*) = 3%.

8.—*Sieve Analysis.*—The sieves and method of making sieve analysis shall conform to the "Standard Method of Test for Sieve Analysis of Aggregates for Concrete" (Serial Designation: C41-24) of the American Society for Testing Materials (Appendix VIII).

9.—*Decantation Test.*—The decantation test shall be made in accordance with the "Tentative Method of Decantation Test for Sand and Other Fine Aggregates" (Serial Designation: D136-22T) of the American Society for Testing Materials (Appendix IX).

10.—*Mortar Strength Test.*—Fine aggregate shall be of such quality that mortar briquettes, cylinders, or prisms, consisting of one part by weight of Portland cement and three parts by weight of fine aggregate,* mixed and tested in accordance with the methods described in the "Standard Specifications and Tests for Portland Cement" (Serial Designation: C9—21) of the American Society for Testing Materials (Appendix II), will show a tensile or compressive strength at ages of 7 and 28 days not less than . . . † per cent. of that of 1:3 standard Ottawa sand mortar of the same plasticity made with the same cement. Concrete tests shall be made in accordance with the "Tentative Methods of Making Compression Tests of Concrete" (Serial Designation: C39—21T) of the American Society for Testing Materials (Appendix XII).

11.—*Organic Impurities in Sand.*—Sand when tested in accordance with the "Standard Method of Test for Organic Impurities in Sands for Concrete"

* In testing aggregate, care should be exercised to avoid the removal of any coating on the grains which may affect the strength. Sand should not be dried before being made into mortar, but should contain its natural moisture. The quantity of water contained may be determined on a separate sample and the weight of the sand used in the test corrected accordingly.

† This percentage must be inserted by the Engineer; it should preferably be 100.

(Serial Designation: C40—22) of the American Society for Testing Materials (Appendix X), shall show a color not darker than the standard color unless it complies with Section 10.

12.—*Permissible Variations.*—Fine aggregate which does not conform to the above requirements for grading, mortar strength, or color, may be used only when approved by the Engineer and then in such proportions as he may require.

C.—Coarse Aggregate*

13.—*General Requirements.*—Coarse aggregates shall consist of crushed stone, gravel, or other approved inert materials with similar characteristics or combinations thereof, having clean, hard, strong, durable, uncoated particles free from injurious amounts of soft, friable, thin, elongated or laminated pieces, alkali, organic, or other deleterious matter.

14.—*Grading.*—Coarse aggregate† shall range in size from fine to coarse within the limits given in Table 2.

TABLE 2.—Size and Grading of Coarse Aggregate.

Passing.	Percentage by weight.
....† in. sieve (maximum size).....	Not less than 95
....† in. " (intermediate size).....	Not less than....†
No. 4 "	Not more than 10
No. 8 "	Not more than 5

† The Engineer must insert in these blanks the sizes and percentages required with regard to materials available. The following table indicates desirable gradings for coarse aggregates of certain nominal maximum sizes:

Nominal maximum size of aggregate, in inches.	Percentage by Weight Passing Through Standard Sieves with Square Openings.						Percentage Passing not more than:	
	3 in.	2 in.	1½ in.	1 in.	¾ in.	½ in.	No. 4 sieve.	No. 8 sieve.
3	95	.....	40-75	.....	.....	.....	10	5
2	.....	95	.....	40-75	.....	.....	10	5
1½	.....	.....	95	.....	40-75	.....	10	5
1	.....	.....	.....	95	.....	.....	10	5
¾	.....	.....	.....	.....	95	.....	10	5
½	.....	.....	.....	.....	.....	95	10	5

15.—*Sieve Sizes.*—The test for size and grading of aggregate shall be made in accordance with the "Standard Method of Test for Sieve Analysis of Aggregates for Concrete" (Serial Designation: C41—24) of the American Society for Testing Materials (Appendix VIII).

16.—*Permissible Variations.*—Coarse aggregate‡ which does not conform to the above requirements, may be used only when approved by the Engineer and then in such proportions as he may require.

* Appendix XVI furnishes a guide in determining the proportions of materials required to produce concrete of a given strength, using aggregates of different sizes and concrete of different consistencies.

† On work of considerable magnitude where several suitable coarse aggregates are available, an investigation of the relative economy of each is advisable.

‡ Requirements for the quality of coarse aggregate for special purposes should be inserted.

D.—Rubble and Cyclopean Aggregate

17.—*Rubble Aggregate.*—Rubble aggregate shall consist of clean, hard, durable stone or gravel larger than 3 in. and weighing not more than 100 lb.

18.—*Cyclopean Aggregate.*—Cyclopean aggregate shall consist of clean, hard, durable stone weighing more than 100 lb.

E.—Storage of Aggregate

19.—*Storage of Aggregate.*—Aggregate shall be so stored as to avoid the inclusion of foreign materials. Frozen aggregate or aggregate containing lumps of frozen material shall be thawed before using.

F.—Water

20.—*General Requirements.*—Water for concrete shall be clean and free from injurious amounts of oil, acid, alkali, organic matter, or other deleterious substance.

G.—Metal Reinforcement

21.—*Quality.*—Metal reinforcement shall meet the requirements of the:

"Standard Specifications* for Billet-Steel Concrete Reinforcement Bars"...* Grade (Serial Designation: A15—14) of the American Society for Testing Materials (Appendix III).

"Standard Specifications* for Rail-Steel Concrete Reinforcement Bars" (Serial Designation: A16—14) of the American Society for Testing Materials (Appendix IV), except that the provision for machining deformed bars before testing shall be eliminated.

22.—*Standard Sizes of Bars.*—Reinforcement bars shall conform to the areas and equivalent sizes shown in Table 3.

TABLE 3.—Sizes and Areas of Reinforcement Bars.†

Size of bar, in inches.	Area, in Square Inches.	
	Round bar.	Square bar.
⅜	0.110	.....
½	0.196	0.250
⅝	0.306	.....
¾	0.441	.....
⅞	0.601	.....
1	0.785	1.000
1⅛	.....	1.265
1¼	.....	1.562

* The Engineer must strike out one of these titles. The Committee recommends, as preferred material for reinforcement, that meeting the requirements of the "Standard Specifications for Billet-Steel Concrete Reinforcement Bars" of intermediate grade, made by the open-hearth process, except that the provisions for machining the test specimens shall be eliminated.

† The Joint Conference of Representatives of Manufacturers, Distributors, and Users of Concrete Reinforcement Bars held in the Department of Commerce, Washington, D. C., September 9, 1924, unanimously adopted the areas and sizes of reinforcement bars in Table 3 with the addition of the ¼ in. round bar, area, 0.049 sq. in.; to become effective as applying to new production, January 1, 1925; every effort to be made to clear current orders and existing stocks of eliminated areas before March 1, 1925.

23.—*Deformed Bars.*—An approved deformed bar shall be one that will develop a bond at least 25% greater than that of a plain round bar of equivalent cross-sectional area.* The areas of deformed bars shall be determined by the minimum cross-section thereof.

24.—*Structural Shapes.*—Structural steel shapes used for reinforcement shall conform to the requirements of the:

"Standard Specifications† for Structural Steel Bridges" (Serial Designation: A7-24) of the American Society for Testing Materials (Appendix V).

"Standard Specifications† for Structural Steel for Buildings" (Serial Designation: A9-24) of the American Society for Testing Materials (Appendix VI).

25.—*Wire.*—Wire for concrete reinforcement shall conform to the requirements of the "Tentative Specifications for Cold-Drawn Steel Wire for Concrete Reinforcement" (Serial Designation: A82-21T) of the American Society for Testing Materials (Appendix VII).

26.—*Cast Iron.*—Cast iron used in composite columns shall conform to the requirements of the "Standard Specifications for Cast-Iron Pipe and Special Castings" (Serial Designation: A44-04) of the American Society for Testing Materials (Appendix XIV).

CHAPTER V

Proportioning and Mixing Concrete

A.—Proportioning

27.—*Unit of Measure.*—The unit of measure shall be the cubic foot. Ninety-four (94) pounds of Portland cement (one bag or ¼ bbl.) shall be considered as one (1) cubic foot.

28.—*Measurement of Aggregates.*—The method of measurement shall be such as to secure the specified proportions in each batch. The aggregate shall be measured separately by volume‡ weight‡.

In volume measurement the fine aggregate and the coarse aggregate shall be measured loose, as thrown into the measuring device, and struck off.§ The water shall be so measured as to insure the desired quantity in successive batches.

29.—*Proportions.*—Concrete shall be mixed in the proportions indicated in Table 4.

Variations in the grading of the aggregates, on which the proportions were based, may be made upon the approval of the Engineer and in such proportions as he may direct (Sections 3 and 14), but no claim shall be made for extra compensation therefor.

* The Committee has under consideration a specification for deformed bars, but is not prepared at this time to make more definite recommendations.

† The Engineer must strike out one of these titles.

‡ The Engineer must strike out one of these terms.

§ The volume of fine aggregate is affected by the method of measurement and by the moisture content; corrections should be made, when necessary, to maintain the required proportions; see, also, footnote to Table 4.

TABLE 4.—PROPORTIONS FOR CONCRETE.*

Part of structure.	PROPORTIONS BY VOLUME.			Concrete strength† assumed as basis for design, in pounds per square inch at 28 days.
	Cement.	Fine aggregate.	Coarse aggregate.	
Footings				‡
Beams, girders, slabs				‡
Columns				‡
				‡
				‡
				‡

* The Engineer should determine, by tests of the available aggregate in advance of use, the proportions necessary to produce concrete of the required strength. Where this is impracticable, Appendix XVI may be used as a guide. Appendix XVI is based on volumes of dry aggregate compacted by rodding in the measure, as specified in "Standard Method of Test for Unit Weight of Aggregate for Concrete," see Appendix XV. Corrections should be made in the quantities in Appendix XVI to take account of the bulking effect of moisture in the fine aggregate. The bulking (swelling) of fine aggregate due to contained moisture, and the method of placing it in the measure, may result in a reduction of 25% in the actual quantity of fine aggregate, as compared with that obtained by dry measurement by the standard method.

† The measure of the quality of concrete is its workability, as determined by the slump test, or other approved test, and its compressive strength of 28 days, using the materials in the proportions specified.

‡ Concrete strengths to be used as basis for design to be filled in by the Engineer.

The Engineer shall have the right to make any changes in proportions or materials that may be necessary or desirable, and proper adjustment of compensation will be made in accordance with the provisions of the contract.

In general, the quantity of water used shall be the minimum necessary to produce concrete of a workability specified in Table 5.

Frequent tests* shall be made throughout the work, as directed by the Engineer, to determine whether the concrete produced by the proportions given in Table 4 is of the quality specified. These tests shall be made in accordance with Section 4 and at the expense of the owner. The Contractor shall provide such facilities as may be practicable for carrying out the tests, and shall cooperate in every way to the end that concrete of the desired quality shall be obtained.

B.—CONSISTENCY OF CONCRETE

30.—*Consistency.*—The quantity of water used shall be the minimum necessary to produce concrete of a workability required by the Engineer.† The consistency of the concrete shall be measured by the slump test as described in "Tentative Method of Test for Consistency of Portland-Cement Concrete for Pavements or for Pavement Base" (Serial Designation: D138-22T) of the American Society for Testing Materials (Appendix XI). The slump for the different types of concrete shall not be greater than that indicated in Table 5, unless authorized by the Engineer.

The consistency shall be checked from time to time during the progress of the work.

* If no tests are to be made on concrete from available materials, or on the concrete mixed during the work, this paragraph must be omitted.

† Attention is called to the fact that increased workability may be obtained by decreasing the quantity of coarse aggregate in the batch, without increasing the quantity of mixing water.

TABLE 5.—WORKABILITY OF CONCRETE AND MORTAR.

Types of Concrete and Mortar.	Maximum slump, in inches.
Mass concrete	†
Reinforced concrete:	
(*a*) Thin vertical sections and columns	†
(*b*) Heavy sections	†
(*c*) Thin confined horizontal sections	†
Roads and pavement:	
(*a*) Hand-finished	†
(*b*) Machine-finished	†
Mortar for floor finish	†

† The Engineer must insert the maximum slumps permitted. The slump-test requirement is intended to insure concrete mixed with the minimum quantity of water required to produce a plastic mixture. The following table indicates the maximum slump desirable for the various types of concrete, based on average aggregates and proportions:

Types of Concrete and Mortar.	Maximum slump, in inches.
Mass concrete	3
Reinforced concrete:	
(*a*) Thin vertical sections and columns	6
(*b*) Heavy sections	3
(*c*) Thin confined horizontal sections	8
Roads and pavements:	
(*a*) Hand-finished	3
(*b*) Machine-finished	1
Mortar for floor finish	2

C.—MIXING

31.—*Machine-Mixing.*—The mixing of concrete, unless otherwise authorized by the Engineer, shall be done in a batch mixer of approved type which will insure a uniform distribution of the materials throughout the mass, so that the mixture is uniform in color and homogeneous. The mixer shall be equipped with suitable charging hopper, water storage, and a water-measuring device controlled from a case which can be kept locked and so constructed that the water can be discharged only while the mixer is being charged. It shall also be equipped with an attachment for automatically locking the discharge lever until the batch has been mixed the required time after all materials are in the mixer. The entire contents of the drum shall be discharged before recharging. The mixer shall be cleaned at frequent intervals while in use. The volume of the mixed material per batch shall not exceed the manufacturer's rated capacity of the mixer.

32.—*Time of Mixing.*—The mixing of each batch shall continue not less than 1 min. after all the materials are in the mixer, during which time the mixer shall rotate at a peripheral speed of about 200 ft. per min.

33.—*Hand-Mixing.*—When hand-mixing is authorized by the Engineer it shall be done on a water-tight platform. The cement and fine aggregate shall first be mixed dry until the whole is of a uniform color. The water and coarse aggregate shall then be added and the entire mass turned at least three (3) times, or until a homogeneous mixture of the required consistency is obtained.

34.—*Retempering.*—The retempering of concrete or mortar which has partly hardened, that is, remixing with or without additional cement, aggregate, or water, will not be permitted.

CHAPTER VI

Depositing Concrete

A.—Depositing in Air

35.—*General.*—Before beginning a run of concrete, hardened concrete and foreign materials shall be removed from the inner surfaces of the mixing and conveying equipment.

36.—*Approval.*—Before depositing concrete, débris shall be removed from the space to be occupied by the concrete; forms shall be thoroughly wetted (except in freezing weather), or oiled. Reinforcement shall be thoroughly secured in position and approved by the Engineer.

37.—*Handling.*—Concrete shall be handled from the mixer to the place of final deposit as rapidly as practicable by methods which prevent the separation or loss of the ingredients. It shall be deposited in the forms as nearly as practicable in its final position to avoid rehandling. It shall be so deposited as to maintain until the completion of the unit, a plastic surface approximately horizontal. Forms for walls or other thin sections of considerable height shall be provided with openings, or other devices, that will permit the concrete to be placed in a manner that will avoid accumulations of hardened concrete on the forms or metal reinforcement. Under no circumstances shall concrete that has partly hardened be deposited in the work.

38.—*Chuting.*—When concrete is conveyed by chuting, the plant shall be of such size and design as to insure a practically continuous flow in the chute. The angle of the chute with the horizontal shall be such as to allow the concrete to flow without separation of the ingredients.* The delivery end of the chute shall be as close as possible to the point of deposit. When the operation is intermittent, the spout shall discharge into a hopper. The chute shall be thoroughly flushed with water before and after each run; the water used for this purpose shall be discharged outside the forms.

39.—*Compacting.*—Concrete, during and immediately after depositing, shall be thoroughly compacted by means of suitable tools. For thin walls or inaccessible portions of the forms, where rodding or forking is impracticable, the concrete shall be assisted into place by tapping or hammering the forms opposite the freshly deposited concrete. The concrete shall be thoroughly worked around the reinforcement, and around embedded fixtures, and into the corners of the forms.

40.—*Removal of Water.*—Water shall be removed from excavations before concrete is deposited, unless otherwise directed by the Engineer. Any flow of water into the excavation shall be diverted through proper side-drains to a sump, or be removed by other approved methods which will avoid washing the freshly deposited concrete. Water vent pipes and drains shall be filled by grouting, or otherwise, after the concrete has thoroughly hardened.

41.—*Protection.*—Exposed surfaces of concrete shall be protected from premature drying for a period of at least seven (7) days after being deposited

* An angle of 27°, or one vertical to two horizontal, is the minimum slope which is considered permissible. Chuting through a vertical pipe is satisfactory when the lower end of the pipe is maintained as nearly as practicable, to the surface of deposit and the pipe full.

42.—*Temperature of Concrete.*—Concrete when deposited shall have a temperature of not less than 40° Fahr., nor more than 120° Fahr. In freezing weather suitable means shall be provided for maintaining the concrete at a temperature of at least 50° Fahr. for not less than 72 hours after placing, or until the concrete has thoroughly hardened. The methods of heating the materials and protecting the concrete shall be approved by the Engineer. Salt, chemicals, or other foreign materials shall not be mixed with the concrete for the purpose of preventing freezing, unless approved by the Engineer.

43.—*Depositing Continuously.*—Concrete shall be deposited continuously and as rapidly as practicable until the unit of operation, approved by the Engineer, is completed. Construction joints at points not provided for in the plans shall be made in accordance with the provisions in Section 69.

44.—*Bonding.*—Before depositing new concrete on or against concrete which has set, the forms shall be re-tightened, the surface of the set concrete shall be roughened as required by the Engineer, thoroughly cleaned of foreign matter and laitance, and saturated with water. The new concrete placed in contact with hardened or partly hardened concrete, shall contain an excess of mortar to insure bond. To insure this excess mortar at the juncture of the hardened and the newly deposited concrete, the cleaned and saturated surfaces of the hardened concrete, including vertical and inclined surfaces, shall first be slushed with a coating of neat cement grout against which the new concrete shall be placed before the grout has attained its initial set.

B.—Rubble and Cyclopean Concrete

45.—*Rubble Concrete.*—Rubble aggregate shall be thoroughly embedded in the concrete. The individual stones shall not be closer to any surface or adjacent stone than the maximum size of the coarse aggregate plus 1 in. Each successive layer of concrete shall be keyed in accordance with the provision in Section 69.

46.—*Cyclopean Concrete.*—Cyclopean aggregate shall be thoroughly embedded in the concrete; no stone shall be closer to a finished surface than 1 ft., nor closer than 6 in. to any adjacent stone. Stratified stone shall be laid on its natural bed.

C.—Depositing Under Water*

47.—*General.*—The methods, equipment, and materials to be used shall be submitted to and be approved by the Engineer before the work is started. Concrete shall be deposited by a method that will prevent the washing of the cement from the mixture, minimize the formation of laitance, and avoid flow of water until the concrete has fully hardened. Concrete shall be placed so as to minimize segregation of materials. Concrete shall not be placed in water having a temperature below 35° Fahr.

* Concrete should not be deposited under water if practicable to deposit in air. There is always uncertainty as to results obtained from placing concrete under water. Where conditions permit, the additional expense and delay of avoiding this method will be warranted. It is especially important that the aggregate be free from loam and other material which may cause laitance. Washed aggregates are preferable. Coarse aggregate consisting of washed gravel of a somewhat smaller size than that used in open-air concrete work will give best results. Concrete should never be deposited under water without experienced supervision. Many failures, especially of structures in sea water, can be traced directly to ignorance of proper methods or lack of expert supervision.

48.—*Proportions.*—Concrete to be deposited under water shall contain 1¾ bbl. (7 bags) or more of Portland cement per cubic yard of concrete in place.

49.—*Coffer-Dams.*—Coffer-dams shall be sufficiently tight to prevent flow of water through the space in which the concrete is to be deposited. Pumping will not be permitted while concrete is being deposited, nor until it has fully hardened.

50.—*Depositing Continuously.*—Concrete shall be deposited continuously, keeping the top surface as nearly level as possible, until it is brought above the water, or to the required height. The work shall be carried on with sufficient rapidity to prevent the formation of layers.

51.—*Method.*—The following method* shall be used for depositing concrete under water:

Tremie.—The tremie shall be water-tight and sufficiently large to permit a free flow of concrete. It shall be kept filled† at all times during depositing. The concrete shall be discharged and spread by raising the tremie in such manner as to maintain as nearly as practicable a uniform flow and avoid dropping the concrete through water. If the charge is lost during depositing the tremie shall be withdrawn and refilled.

Drop-Bottom Bucket.—The bucket shall be of a type that cannot be dumped until it rests on the surface upon which the concrete is to be deposited. The bottom doors when tripped shall open freely downward and outward. The top of the bucket shall be open. The bucket shall be completely filled, and slowly lowered to avoid back-wash. When discharged, the bucket shall be withdrawn slowly until well above the concrete.

Bags.—Bags of jute or other coarse cloth shall be filled about two-thirds (⅔) full of concrete and carefully placed by hand in a header-and-stretcher system so that the whole mass is interlocked.

52.—*Laitance.*—Great care shall be exercised to disturb the concrete as little as possible when it is being deposited in order to avoid the formation of laitance. On completing a section of concrete, the laitance shall be entirely removed before work is resumed.

CHAPTER VII

Forms

53.—*General.*—Forms shall conform to the shape, lines, and dimensions of the concrete as called for on the plans. Lumber used in forms for exposed surfaces shall be dressed to a uniform thickness, and shall be free from loose knots or other defects. Joints in forms shall be horizontal or vertical. For unexposed surfaces and rough work undressed lumber may be used. Lumber once used in forms shall have nails withdrawn and surfaces to be in contact with concrete thoroughly cleaned before being used again.

* The Engineer must strike out the method or methods not applicable to the work.

† The tremie may be filled by one of the following methods: (1) Place the lower end in a box partly filled with concrete, so as to seal the bottom, then lower into position; (2) plug the tremie with cloth sacks or other material, which will be forced down as the pipe is filled with concrete; (3) plug the end of the tremie with cloth sacks filled with concrete.

54.—*Design.*—Forms shall be substantial and sufficiently tight to prevent leakage of mortar; they shall be properly braced or tied together so as to maintain position and shape. If adequate foundation for shores cannot be secured, trussed supports shall be provided.

55.—*Workmanship.*—Bolts and rods shall preferably be used for internal ties; they shall be so arranged that when the forms are removed no metal shall be within 1 in. of any surface. Wire ties will be permitted only on light and unimportant work; they shall not be used through surfaces where discoloration would be objectionable. Shores supporting successive stories shall be placed directly over those below, or so designed that the load will be transmitted directly to them. Forms shall be set to line and grade and so constructed and fastened as to produce true lines. Special care shall be used to prevent bulging.

56.—*Mouldings.*—Unless otherwise specified suitable mouldings or bevels shall be placed in the angles of forms to round or bevel the edges of the concrete.

57.—*Oiling.*—The inside of forms shall be coated with non-staining mineral oil or other approved material, or thoroughly wetted (except in freezing weather). Where oil is used, it shall be applied before the reinforcement is placed.

58.—*Inspection.*—Temporary openings shall be provided at the base of column and wall forms and at other points where necessary to facilitate cleaning and inspection immediately before depositing concrete.

59.—*Removal.*—Forms shall not be disturbed until the concrete has adequately hardened.* Shoring shall not be removed until the member has acquired sufficient strength to support safely its weight and the load on it. Members subject to additional loads during construction shall be adequately shored to support both the member and construction loads in such a manner as will protect the member from damage by the loads; this shoring shall not be removed until the member has acquired sufficient strength to support safely its weight and the load on it.

CHAPTER VIII

Details of Construction

A.—Metal Reinforcement

60.—*Cleaning.*—Metal reinforcement, before being positioned, shall be thoroughly cleaned of mill and rust scale and of coatings that will destroy or reduce the bond. Reinforcement appreciably reduced in section shall be rejected. Where there is delay in depositing concrete, reinforcement shall be reinspected and, when necessary, cleaned.

61.—*Bending.*—Reinforcement shall be carefully formed to the dimensions indicated on the plans, or as provided in Section 140. Cold bends shall be made around a pin having a diameter of four (4) or more times the least dimension of the reinforcement bars for steel of structural grade and eight (8) or more times that for steel of intermediate or hard grade.

* Many conditions affect the hardening of concrete, and the proper time for the removal of the forms should be determined by the Engineer.

62.—*Straightening.*—Metal reinforcement shall not be bent or straightened in a manner that will injure the material. Bars with kinks or bends not shown on the plans shall not be used. Heating of reinforcement will be permitted only when the entire operation is approved by the Engineer.

63.—*Placing.*—Metal reinforcement shall be accurately positioned, and secured against displacement by using annealed iron wire of not less than No. 18 gauge, or suitable clips at intersections, and shall be supported by concrete or metal chairs or spacers, or metal hangers. The minimum clear distance between parallel bars shall be one and one-half (1½) times the diameter of round bars, or one and one-half (1½) times the diagonal of square bars; if the ends of bars are anchored as specified in Section 140, the clear spacing may be made equal to the diameter of round bars, or to the diagonal of square bars, but in no case shall the spacing between bars be less than 1 in., nor less than one and one-quarter (1¼) times the maximum size of the coarse aggregate. Bars parallel to the face of any member shall be embedded a clear distance of not less than one (1) diameter from the face.

64.—*Splicing.*—In slabs, beams, and girders, splices of reinforcement shall not be made at points of maximum stress without the approval of the Engineer. Splices, where permitted, shall provide sufficient lap to transfer the stress between bars by bond and shear. In such splices the bars shall be placed at the minimum distance specified in Section 63; adjacent bars shall not be spliced at the same point.

Splices in columns, piers, and struts shall provide sufficient lap to transfer the stress by bond.

65.—*Offsets in Reinforcement of Compression Members.*—Where changes in the cross-section of a compression member occur, the longitudinal reinforcement bars shall be sloped for the full length of the member, or offset in a region where lateral support is afforded. Where offset, the slope of the inclined portion from the axis of the member shall not be more than 1 in 6.

66.—*Future Bonding.*—Exposed reinforcement bars intended for bonding with future extensions shall be protected from corrosion.

B.—Protective Concrete Covering

67.—*Moisture Protection.*—Metal reinforcement in wall footings and column footings shall have a minimum covering of 3 in. of concrete. At surfaces of concrete exposed to the weather, metal reinforcement shall be protected by not less than 2 in. of concrete.

68.—*Fire Protection.*—Metal reinforcement in fire-resistive construction shall be protected by not less than 1 in. of concrete in slabs and walls, and not less than 2 in. in beams, girders, and columns, provided aggregate showing an expansion not materially greater than that of limestone or trap-rock is used; when impracticable to obtain aggregate of this grade, the protective covering shall be 1 in. thicker and shall be reinforced with metal mesh, having openings not exceeding 3 in., placed 1 in. from the finished surface.

In structures where the fire hazard is limited, the metal reinforcement shall not be placed nearer the exposed surface than ¾ in. in slabs and walls, or 1½ in. in beams, girders, and columns.

C.—Joints

69.—*General.*—Joints not indicated on the plans shall be so designed and located as least to impair the strength and appearance of the structure. To prevent laitance in horizontal joints, excess water shall be removed from the surface forming the joint after depositing the concrete. Surfaces of contact shall be cleaned and wetted before depositing is resumed, and any laitance shall be removed. Where additional resistance to horizontal shear is required, stones shall be partly embedded in such a manner as to key with the adjoining concrete; or mortices or keys shall be formed in the concrete.

70.—*Columns.*—Joints in columns shall be made at the under side of the floor. Haunches and column capitals shall be considered as part of, and to act continuous with the floor. At least two (2) hours must elapse after depositing concrete in the columns or walls before depositing in beams, girders, or slabs.

71.—*Floors.*—Construction joints in floors shall be located near the middle of span of slabs, beams, or girders, unless a beam intersects a girder at this point, in which case the joints in the girders shall be offset a distance equal to twice the width of the beam. Adequate provision shall be made for shear by use of inclined reinforcement.

72.—*Construction Joints in Long Buildings.*—Construction joints made cross-wise of a building 100 ft. or more in length, shall have special reinforcement placed at right angles to the joint, and extending a sufficient distance on each side of the joint, to develop the strength of the reinforcement by bond. This reinforcement shall be placed near the opposite face of the member from the main tension reinforcement; the cross-sectional area of such reinforcement shall be not less than 0.5% of the section of the members cut by the joint.

73.—*Expansion Joints.*—Expansion joints shall be so detailed that the necessary movement may occur with minimum resistance at the joint. The structure adjacent to the joint shall preferably be supported on separate columns or walls. Reinforcement shall not extend across an expansion joint; the break between the two sections shall be complete.* Exposed edges of expansion joints in walls or abutments shall be rounded. Exposed expansion joints between two distinct concrete members shall be filled with an elastic joint filler of approved quality.

74.—*Expansion Joints in Long Buildings.*—Buildings exceeding 200 ft. in length and of width less than about one-half (½) the length, shall be divided by means of expansion joints, located near the middle, but not more than 200 ft. apart, to minimize the destructive effects of temperature changes and shrinkage. Where there is an abrupt change in the width of a building, an expansion joint shall be provided.

75.—*Sliding Joints.*—The seat of sliding joints shall be finished to a smooth plane surface and allowed to harden. Two thicknesses of building paper shall be placed on the seat before depositing superimposed concrete.†

* A coating of white lead and oil, asphalt paint, petrolatum, or water-proofed building paper placed over the entire surface of the hardened concrete, is commonly used for this purpose.

† Sheet zinc, lead, and bronze are also used for this purpose.

76.—*Water-Tight Construction Joints.*—Where construction joints are required to be water-tight the method of construction shall be as follows:

(*a*) Horizontal joints shall be constructed by forming a continuous keyway in the lower portion of concrete before the concrete has hardened. Before placing the superimposed concrete, the joint shall be thoroughly cleaned of laitance or other foreign material, saturated with water, and coated with neat cement grout. The superimposed concrete shall be placed in such a manner as will insure an excess of mortar over the entire surface of the joint.

(*b*) Vertical joints shall be made by a metal water-stop approved by the Engineer.

Seepage water shall be collected and drained from the forms; where required, vent pipes shall be closed after the concrete has thoroughly hardened.*

CHAPTER IX

Water-Proofing and Protective Treatment

A.—Water-Proofing

77.—*General.*—Concrete required to be water-tight shall be made with strict adherence to all provisions in these specifications regarding the choice of materials, proportions, consistency, mixing, placing, protecting, and workmanship.

78.—*Integral Compounds.*—Integral compounds shall not be used for water-proofing unless specifically authorized by the Engineer.

79.—*Water-tight Joints.*—See Section 76.

B.—Oil-Proofing

80.—*Oil-Proofing.*—Concrete containers for light mineral oils, animal oils, certain vegetable oils, and other commercial liquids shall be given an inside coating which shall be applied before the container is placed in service; the coating and the method of application shall be approved by the Engineer.† Floors or other surfaces exposed to heavy concentrations of such oils or liquids shall be similarly protected.

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* To secure water-tight joints it is vitally essential that all incoming water be drained from behind the forms during the process of concreting. If the upper portion of the joint is concreted first, a metal water-stop should be provided as directed by the Engineer.

† The effect of certain oils and other liquids on concrete and surface treatments which have been found beneficial are given in Appendix XVII.

Concrete containers for mineral oils of 30° Baumé and lighter should be given a coating which will not be affected or penetrated by mineral spirits. Coatings containing ingredients which saponify or oxidize in the presence of lime shall be applied only to a surface which has previously been treated to neutralize the lime.

Concrete containers for mineral oils heavier than 30° Baumé require no coating.

Certain vegetable and animal oils and strong acids and alkaline solutions have a destructive effect on concrete and some protective treatment is required. Several vegetable oils have a disintegrating effect on concrete in cases where the surface is alternately wet and dry, yet when stored in closed concrete containers have no injurious action.

Concrete containers for commercial liquids involve certain special features in addition to the general requirements of good concrete. Containers for such liquids often have thin sections, therefore a rich mixture is necessary to obtain concrete of proper workability without sacrificing strength and impermeability; it is especially necessary to eliminate joints or seams; thorough spading is essential and should be continuous during the placing of concrete.

C.—Concrete in Sea Water

81.—*Proportions.*—Plain concrete in sea water from 2 ft. below low water to 2 ft. above high water, or from a plane below to a plane above wave action, shall contain a minimum of 1¾ bbl. (7 bags) of Portland cement per cubic yard in place. Other plain concrete in sea water or exposed directly along the sea coast shall contain a minimum of 1½ bbl. (6 bags) of Portland cement per cubic yard in place. Porous or weak aggregates shall not be used.

82.—*Consistency.*—The consistency shall meet the requirements of Section 30.

83.—*Depositing.*—Sea water shall not be allowed to come in contact with the concrete until it has hardened for at least four (4) days. Concrete shall be placed in such a manner as to minimize the number of horizontal or inclined seams or work planes. The placing of concrete between tides shall be a continuous operation, in accordance with the methods described in Section 43; where it is impossible to avoid seams or joints proceed as in Section 44. Concrete shall be deposited in sea water only when so directed by the Engineer, in which case it shall be placed in accordance with the methods described in Sections 47 to 52.

84.—*Protection.*—Metal reinforcement shall be placed at least 3 in. from any plane or curved surface except at corners when it shall be at least 4 in. from adjacent surfaces. Metal chairs, supports or ties shall not extend to the surface of the concrete. Where unusually severe conditions of abrasion are anticipated, the face of the concrete from 2 ft. below low water to 2 ft. above high water, or from a plane below to a plane above wave action, shall be protected by creosoted timber, dense vitrified shale brick, or stone of suitable quality, as designated on the plans or as required by the Engineer.

D.—Concrete in Alkali Soils or Waters*

85.—*Proportions.*—Concrete in alkali waters or below the ground-line of alkali soils shall contain a minimum of 1¾ bbl. (7 bags) of Portland cement per cubic yard in place.†

86.—*Consistency.*—The consistency of concrete in alkali soils or waters shall be such as to meet the requirements of Section 30.

87.—*Placing.*—Concrete shall be placed in such a manner as to minimize the number of horizontal or inclined seams, or work planes.

88.—*Protection.*—Metal reinforcement or other corrodible metal shall not be placed closer than 2 in. to the surface of members exposed to alkali soils or

* Under certain circumstances concrete is attacked by alkaline waters. The term "alkali" is here used to designate the soluble salts which occur in considerable quantities in the soils and waters of certain Western States; the sulphates, chlorides, and carbonates of sodium and magnesium are the most common forms of alkali.

It is important to distinguish between the different forms of alkali which occur in Nature. Experience and tests have shown that certain forms are more injurious than others; the chlorides and carbonates produce little or no injury to concrete.

Special care must be used in placing concrete where it will be exposed to sulphate waters. An impermeable concrete made with a durable aggregate is necessary. Concrete should be permitted to harden under favorable conditions before it is exposed to injurious alkalies, and wherever practicable such concrete should be made in the form of precast units.

† Where the foundations of important buildings or similar structures are subject to high concentrations of alkalies, under-drainage may be used as an added precaution.

waters. In foundations and in heavy structures the metal reinforcement shall not be placed closer than 3 in. from the surface.

CHAPTER X

Surface Finish*

89.—*General.*—The requirements in these specifications applying to forms, and to mixing, conveying, depositing, and finishing concrete, shall be followed unless modified by the plans.

The whole of a showing face between prescribed construction joints shall be cast in one continuous operation. Construction joints, when not shown on the plans, shall be made as directed by the Engineer, and shall be true to line with sharp unbroken edges.

The same brand of cement, and the same kind and size of aggregate, shall be used throughout the whole of any showing face.

For showing faces the forms shall be smooth and water-tight. If wood be used, the boards shall be planed, grooved, and tongued, evenly matched, and tightly placed. They shall be so constructed as to be removable in sections by unscrewing or otherwise loosening them without hammering or prying against the face. Any offsets in the forms that may occur shall be smoothly dressed and any openings pointed flush with stiff clay or plaster of Paris in order to prevent leakage or the formation of fins.

Concrete that is to have a showing face, whether any particular finish is called for or not, shall be mixed, placed, and worked as may be necessary to secure at the face a uniform distribution of the aggregates, freedom from void spaces, and uniform texture. If the finish is required to be one that will expose the coarse aggregate, by either scrubbing, tooling, sand-blasting, or acid treatment, then after the full surface of mortar has been worked against the form, the spading tool shall be inserted in the concrete and the coarse aggregate be pressed against the form, in order to secure uniform distribution at the face and a uniform texture after the aggregate is exposed.

Face forms shall be removed as soon as practicable in order to facilitate effective repair of void spaces or broken corners, before the surface has dried. Care shall be taken to avoid roughening or injuring corners, and to keep edges sharp.

As soon as the face forms are removed any fins or other projections shall be carefully removed, and offsets leveled, and any voids or damaged places shall immediately be saturated with water and filled with a mixture of the same composition as that used in the surface, and brought even with the surface by means of a wooden spatula or float. A steel trowel shall not be used to finish the surface. The face shall be finished free from streaks, discolorations, or other imperfections. Plastering will not be permitted.

Where a surface of mortar is to be the basis of the finish the coarse aggregate shall be worked back from the form with a suitable tool, so as to bring a full surface of mortar against the form, care being taken to prevent the formation of voids and air-pockets.

* The Engineer should indicate on the plans the type of finish desired and must strike out the sections which do not apply.

Whenever forms are removed from showing faces before the concrete has become hard and dry, the surface of the concrete shall be immediately wetted and kept wet for at least three (3) days.

90.—*Granolithic Surfaces.*—Granolithic surfaces shall be made by placing about 1 in. of facing concrete against the face form in advance of the concrete, of such consistency and in such a manner as will insure its bonding with the concrete.

The facing concrete shall be composed of 1 part of Portland cement, 1½ parts of fine aggregate, and 2½ parts of coarse aggregate made up of pebbles, crushed granite, or other stone as called for.

If iron or wooden moulds are used to retain the facing against the forms while placing concrete, care should be taken that the mould is not permitted to remain until initial setting occurs. The moulds shall be jarred frequently and raised at short intervals to prevent formation of seams and air spaces between the surface and the concrete.

91.—*Top Surfaces not Subject to Wear.*—Top surfaces not subject to wear shall be smoothed with a wood float and be kept wet for at least seven (7) days. Care shall be taken to avoid an excess of water in the concrete, and to drain or otherwise promptly remove any water that comes to the surface. Dry cement, or a dry mixture of cement and sand, shall not be sprinkled directly on the surface.

A.—Wearing Surfaces

92.—*One-Course Work.*—Aggregates for the wearing surface shall have a high resistance to abrasion, and shall be screened and when necessary thoroughly washed. The least quantity of mixing water that will produce a dense concrete shall be used. The mix shall not be leaner than 1 part of Portland cement and 2½ parts of aggregate. The surface shall be screened even and finished with a wood float. Excess water shall be promptly drained or otherwise removed. Over-troweling shall be avoided.

93.—*Two-Course Work.*—The wearing surface in two-course work shall be placed within one-half (½) hour after the base course. Where the wearing surface is required to be applied to a hardened base course, the latter shall be prepared by roughening with a pick or other effective tool. The roughened surface shall be thoroughly saturated with water and covered with a thin layer of neat cement paste immediately before the wearing surface is placed. The wearing course shall not be thinner than 1 in.

94.—*Curing.*—Concrete wearing surfaces made in accordance with Sections 92 to 93 shall be kept wet* for at least ten (10) days in the case of floors and twenty-one (21) days in the case of roads and pavements.

95.†—*Terrazzo Finish.*—Terrazzo finish shall be made by mixing 1 part of cement, 2½ parts of crushed marble, or other stone or crushed pebbles, as may be called for by the plans, and sufficient water to produce a dense concrete. The concrete shall be spread on the base course and worked down to a thick-

* Prevention of premature drying during the early hardening of concrete is essential to the development of high resistance to abrasion. The surface may be covered with a layer of burlap, earth, or sand, kept wet, or it may be divided into small areas by dikes and flooded with water to a depth of 2 or 3 in.

† The Engineer must strike out one of the Sections 95, pp. 1185 and 1186.

ness of 1 in. by patting or rolling and troweling. The marble shall all pass a ½-in. screen and be free from dust. The surface shall be kept wet for not less than ten (10) days, and after curing shall be rubbed to a plane surface with a stone or a surfacing machine. Hardened concrete to which a terrazzo finish is to be applied shall be prepared as prescribed in Section 93.

95.*—*Terrazzo Finish.*—Terrazzo finish shall be made by mixing 1 part of cement, 2 parts of sand, and sufficient water to produce a plastic mortar, which shall be spread on the base course to a depth of 1 in. Crushed marble, free from dust and passing a ¼-in. screen, shall be sprinkled over the surface of the fresh mortar, and pressed or rolled in. The surface shall be kept wet for not less than ten (10) days and after curing shall be rubbed to a plane surface with a stone or a surfacing machine. Hardened concrete to which a terrazzo finish is to be applied shall be prepared as prescribed in Section 93.

B.—Decorative Finishes

96.—*Rubbed Finish.*—Immediately after the forms are removed, the surface shall be wetted and rubbed with a carborundum brick or other abrasive until even and smooth and of uniform appearance, without applying any cement or other coating.

97.—*Scrubbed Finish.*—The forms must be removed and the scrubbing done before the concrete has hardened.† The surface shall be scrubbed with a fiber or wire brushes using water freely, until the surface film of mortar is removed and the aggregate uniformly exposed; then rinsed with clean water. If portions of the surface have become too hard to scrub in equal relief, dilute hydro-chloric acid (1 part acid to 4 parts water) may be used to facilitate the scrubbing. The remaining acid shall be thoroughly removed with clean water.

98.—*Sand-Blast Finish.*—The concrete face shall be permitted to attain an intermediate degree of hardness; it shall then be air-blasted with hard sand until the aggregate is in uniform relief.

99.—*Tooled Finish.*—The surface shall be permitted to become dry and hard, and then dressed with tools, as called for,‡ to a uniform texture and even face.

100.—*Sand-Floated Finish.*—The forms shall be removed before the surface has fully hardened; the surface shall be rubbed with a wooden float by a uniform circular motion, fine sand being rubbed into the surface until the resulting finish is even and uniform.

101.—*Colored Aggregate Finish.*—Colored or other special aggregate used for finish shall be exposed by scrubbing as provided in Section 97. Facing mortar, made from this special aggregate, of 1 part of cement, 1½ parts of sand, and 3 parts of pebbles or screenings, clean and free from dust, shall be placed against the face forms to a thickness of about 1 in. sufficiently in advance of the concrete to prevent the latter coming in contact with the form.

102.—*Colored Pigment Finish.*—Mineral pigment shall be thoroughly mixed dry with the cement and fine aggregate; care shall be taken to secure a uniform tint throughout.

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* The Engineer must strike out one of the Sections 95, pp. 1185 and 1186.

† In warm weather this will require from 6 to 24 hours, and in cold weather from 1 to 3 days.

‡ By axing, rough or fine pointing, or bush-hammering.

CHAPTER XI

DESIGN

A.—GENERAL ASSUMPTIONS

103.—*General Assumptions.*—The design of reinforced concrete members under these specifications shall be based on the following assumptions:

(*a*) Calculations are made with reference to working stresses and safe loads rather than with reference to ultimate strength and ultimate loads.

(*b*) A plane section before bending remains plane after bending, shearing distortions being neglected.

(*c*) The modulus of elasticity of concrete in compression is constant within the limits of working stresses and the distribution of compressive stress in beams is rectilinear.

(*d*) The moduli of elasticity of concrete in computations for the position of the neutral axis, for the resisting moment of beams, and for compression of concrete in columns, are as follows:

1.—One-fifteenth ($\frac{1}{15}$) that of steel, when the compressive strength of the concrete at 28 days exceeds 1 500 and does not exceed 2 200 lb. per sq. in.

2.—One-twelfth ($\frac{1}{12}$) that of steel, when the compressive strength of the concrete at 28 days exceeds 2 200 and does not exceed 2 900 lb. per sq. in.

3.—One-tenth ($\frac{1}{10}$) that of steel, when the compressive strength of the concrete at 28 days is greater than 2 900 lb. per sq. in.

(*e*) In calculating the moment of resistance of reinforced concrete beams and slabs the tensile resistance of the concrete is neglected.

(*f*) The bond between the concrete and the metal reinforcement remains unbroken throughout the range of working stresses. Under compression the two materials are therefore stressed in proportion of their moduli of elasticity.

(*g*) Initial stress in the reinforcement due to contraction or expansion of the concrete is neglected, except in the design of reinforced concrete columns.*

B.—FLEXURE OF RECTANGULAR REINFORCED CONCRETE BEAMS AND SLABS

104.—*Flexure Formulas.*—Computations of flexure in rectangular reinforced concrete beams and slabs shall be based on the following formulas:

(*a*) *Reinforced for Tension Only:*

Position of neutral axis,

$$k = \sqrt{2\,p\,n + (p\,n)^2} - p\,n \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

Arm† of resisting couple,

$$j = 1 - \frac{k}{3} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

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* Formula (43) for the permissible compressive stress in reinforced concrete columns takes into account the effect of shrinkage in the concrete on the stress in the longitudinal reinforcement. It is not required, however, that the designer consider shrinkage stresses in columns, except through the use of that formula.

† For $f_s = 16\,000$ to $18\,000$ lb. per sq. in. and $f_c = 800$ to 900 lb. per sq. in., j may be assumed as 0.86. For values of $p\,n$ varying from 0.04 to 0.24, $j\,k$ is approximately equal to $0.67\,\sqrt[3]{p\,n}$.

Compressive unit stress* in extreme fiber of concrete,

$$f_c = \frac{2\,M}{j\,k\,b\,d^2} = \frac{2\,p\,f_s}{k} \quad \ldots\ldots (3)$$

Tensile unit stress* in longitudinal reinforcement,

$$f_s = \frac{M}{A_s\,j\,d} = \frac{M}{p\,j\,b\,d^2} \quad \ldots\ldots (4)$$

Steel ratio for balanced reinforcement,

$$p = \frac{1}{2}\,\frac{1}{\frac{f_s}{f_c}\left(\frac{f_s}{n\,f_c} + 1\right)} \quad \ldots\ldots (5)$$

For formulas on shear and bond, see Sections 121 to 127 and 135 to 137.

(*b*) *Reinforced for Both Tension and Compression:*

Position of neutral axis,

$$k = \sqrt{2\,n\left(p + p'\,\frac{d'}{d}\right) + n^2\,(p + p')^2} - n\,(p + p') \quad \ldots\ldots (6)$$

Position of resultant compression,

$$z = \frac{\frac{1}{3}\,k^3\,d + 2\,p'\,n\,d'\left(k - \frac{d'}{d}\right)}{k^2 + 2\,p'\,n\left(k - \frac{d'}{d}\right)} \quad \ldots\ldots (7)$$

Arm* of resisting couple,

$$j\,d = d - z \quad \ldots\ldots (8)$$

Compressive unit stress in extreme fiber of concrete,

$$f_c = \frac{6\,M}{b\,d^2\left[3\,k - k^2 + \frac{6\,p'\,n}{k}\left(k - \frac{d'}{d}\right)\left(1 - \frac{d'}{d}\right)\right]} \quad \ldots\ldots (9)$$

Tensile unit stress in longitudinal reinforcement,

$$f_s = \frac{M}{p\,j\,b\,d^2} = n\,f_c\,\frac{1 - k}{k} \quad \ldots\ldots (10)$$

Compressive unit stress in longitudinal reinforcement,

$$f'_s = n\,f_c\,\frac{k - \frac{d'}{d}}{k} \quad \ldots\ldots (11)$$

105.—*Notation.*—The symbols† used in Formulas (1) to (23) are defined as follows:

A_s = effective cross-sectional area of metal reinforcement in tension in beams;

b = width of rectangular beam or width of flange of T-beam;

d = depth from compression surface of beam or slab to center of longitudinal tension reinforcement;

d' = depth from compression surface of beam or slab to center of compression reinforcement;

* For f_s = 16 000 to 18 000 lb. per sq. in. and f_c = 800 to 900 lb. per sq. in., j may be assumed as 0.86. For values of $p\,n$ varying from 0.04 to 0.24, $j\,k$ is approximately equal to 0.67 $\sqrt[3]{p\,n}$.

† For illustration of notation as applied to typical beams or slabs, see Figs. 1 and 2 (Appendix I).

f_c = compressive unit stress in extreme fiber of concrete;
f_s = tensile unit stress in longitudinal reinforcement;
f'_s = compressive unit stress in longitudinal reinforcement;
h = unsupported length of column;
I = moment of inertia of a section about the neutral axis for bending;
j = ratio of lever arm of resisting couple to depth, d;
k = ratio of depth of neutral axis to depth, d;
l = span length of beam or slab (generally distance from center to center of supports, see Section 106);
M = bending moment or moment of resistance in general;
$n = \frac{E_s}{E_c}$ = ratio of modulus of elasticity of steel to that of concrete;
p = ratio of effective area of tension reinforcement to effective area of concrete in beams $= \frac{A_s}{b\,d}$;
p' = ratio of effective area of compression reinforcement to effective area of concrete in beams;
w = uniformly distributed load per unit of length of beam or slab;
z = depth from compression surface of beam or slab of resultant of compressive stresses.

106.—*Span Length.*—The span length, l, of freely supported beams and slabs shall be the distance between centers of the supports, but shall not exceed the clear span plus the depth of beam or slab. The span length for continuous or restrained beams built to act integrally with supports shall be the clear distance between faces of supports. Where brackets having a width not less than the width of the beam and making an angle of 45° or more with the horizontal axis of a restrained beam are built to act integrally with the beam and support, the span shall be measured from the section where the combined depth of the beam and bracket is at least one-third ($\frac{1}{3}$) more than the depth of the beam, but no portion of such a bracket shall be considered as adding to the effective depth of the beam. Maximum negative moments are to be considered as existing at the ends of the span, as defined above.

107.—*Slightly Restrained Beams of Equal Span.*—Beams and slabs of equal spans built to act integrally with beams, girders, or other slightly restraining supports and carrying uniformly distributed loads shall be designed for the following moments at critical sections:

(*a*) Beams and slabs of one span,
Maximum positive moment near center,

$$M = \frac{w\,l^2}{8} \qquad (12)$$

(*b*) Beams and slabs continuous for two spans only,
1.—Maximum positive moment near center,

$$M = \frac{w\,l^2}{10} \qquad (13)$$

2.—Negative moment over interior support,

$$M = \frac{w\,l^2}{8} \qquad (14)$$

(*c*) Beams and slabs continuous for more than two spans,

1.—Maximum positive moment near center and negative moment at support of interior spans,

$$M = \frac{w\,l^2}{12} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

2.—Maximum positive moment near centers of end spans and negative moment at first interior support,

$$M = \frac{w\,l^2}{10} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

(*d*) Negative moment at end supports for Cases (*a*), (*b*), and (*c*) of this Section,

$$M = \text{not less than } \frac{w\,l^2}{16} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (16a)$$

108.—*Beams Built into Brick or Masonry Walls.*—Beams and slabs built into brick or masonry walls in a manner which develops partial end restraint shall be designed for a negative moment at the support of,

$$M = \text{not less than } \frac{w\,l^2}{16} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (17)$$

109.—*Freely Supported Beams of Equal Span.*—Beams and slabs of equal spans freely supported and assumed to carry uniformly distributed loads shall be designed for the moments specified in Section 107, except that no reinforcement for negative moment need be provided at end supports where effective measures are taken to prevent end restraint. The span shall be taken as defined in Section 106 for freely supported beams.

110.—*Restrained Beams of Equal Span.*—Beams and slabs of equal span built to act integrally with columns, walls, or other restraining supports and assumed to carry uniformly distributed loads, shall be designed (except as provided in Section 107) for the following moments at critical sections:

(*a*) Interior spans,

1.—Negative moment at interior supports except the first,

$$M = \frac{w\,l^2}{12} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (18)$$

2.—Maximum positive moment near centers of interior spans,

$$M = \frac{w\,l^2}{16} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (19)$$

(*b*) End spans of continuous beams and beams of one span in which $\frac{I}{l}$ is less than twice the sum of the values of $\frac{I}{h}$ for the exterior columns above and below which are built into the beams:

1.—Maximum positive moment near center of span and negative moment at first interior supports,

$$M = \frac{w\,l^2}{12} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (20)$$

2.—Negative moment at exterior supports,

$$M = \frac{w\,l^2}{12} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (21)$$

(*c*) End spans of continuous beams, and beams of one span, in which $\frac{I}{l}$ is equal to or greater than twice the sum of the values of $\frac{I}{h}$ for the exterior columns above and below which are built into the beam:

1.—Maximum positive moment near center of span and negative moment at first interior support,

$$M = \frac{w\,l^2}{10} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (22)$$

2.—Negative moment at exterior support,

$$M = \frac{w\,l^2}{16} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (23)$$

111.—*Continuous Beams of Unequal Spans or with Non-Uniform Loading.*—Continuous beams with unequal spans, or with other than uniformly distributed loading, whether freely supported or restrained, shall be designed for the actual moments under the conditions of loading and restraint.

Provision shall be made where necessary for negative moment near the center of short spans which are adjacent to long spans, and for the negative moment at the end supports, if restrained.

112.—*Unsupported Flange Length.*—The distance between lateral supports of the compression area of a beam shall not exceed twenty-four (24) times the least width of compression flange.

C.—Flexure of Reinforced Concrete T-Beams

113.—*Flexure Formulas.*—Computations of flexure in reinforced concrete T-beams shall be based on the following formulas:

(*a*) *Neutral Axis in the Flange:*

Use formulas for rectangular beams and slabs in Section 104.

(*b*) *Neutral Axis below the Flange:*

Position of neutral axis,

$$k\,d = \frac{2\,n\,d\,A_s + b\,t^2}{2\,n\,A_s + 2\,b\,t} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (24)$$

Position of resultant compression,

$$z = \left(\frac{3\,k\,d - 2\,t}{2\,k\,d - t}\right)\frac{t}{3} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (25)$$

Arm of resisting couple,

$$j\,d = d - z \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (26)$$

Compressive unit stress in extreme fiber of concrete,

$$f_c = \frac{M\,k\,d}{b\,t\,(k\,d - \frac{1}{2}\,t)\,j\,d} = \frac{f_s}{n}\left(\frac{k}{1-k}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (27)$$

Tensile unit stress in longitudinal reinforcement,

$$f_s = \frac{M}{A_s j\, d} \qquad (28)$$

Formulas (24), (25), (26), (27), and (28) neglect compression in the stem.*

114.—*Notation.*—The symbols† used in Formulas (24) to (28) are defined in Section 105 except as follows:

b' = width of stem of T-beam;

t = thickness of flange of T-beam.

115.—*Flange Width.*—Effective and adequate bond and shear resistance shall be provided in beam-and-slab construction at the junction of the beam and slab; the slab shall be built and considered an integral part of the beam; the effective flange width to be used in the design of symmetrical T-beams shall not exceed one-fourth ($\frac{1}{4}$) of the span length of the beam, and its overhanging width on either side of the web shall not exceed eight (8) times the thickness of the slab nor one-half ($\frac{1}{2}$) the clear distance to the next beam.

For beams having a flange on one side only, the effective flange width to be used in design shall not exceed one-tenth ($\frac{1}{10}$) of the span length of the beam, and its overhanging width from the face of the web shall not exceed six (6) times the thickness of the slab nor one-half ($\frac{1}{2}$) the clear distance to the next beam.

116.—*Transverse Reinforcement.*—Where the principal slab reinforcement is parallel to the beam, transverse reinforcement, not less in amount than 0.3% of the sectional area of the slab, shall be provided in the top of the slab and shall extend across the beam and into the slab not less than two-thirds ($\frac{2}{3}$) of the width of the effective flange overhang. The spacing of the bars shall not exceed 18 in.

117.—*Compressive Stress at Supports.*—Provision shall be made for the compressive stress at the support in continuous T-beam construction.

118.—*Shear.*—The flange of the beam shall not be considered as effective in computing the shear and diagonal tension resistance of T-beams.

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* The following formulas take into account the compression in the stem; they are recommended where the flange is small compared with the stem:

Position of neutral axis,

$$k\,d = \sqrt{\frac{2\,n\,d\,A_s + (b-b')\,t^2}{b'} + \left(\frac{n\,A_s + (b-b')\,t}{b'}\right)^2} - \frac{n\,A_s + (b-b')\,t}{b'} \qquad (24a)$$

Position of resultant compression,

$$z = \frac{(k\,d\,t^2 - \frac{2}{3}\,t^3)\,b + [(k\,d-t)^2\,(t + \frac{1}{3}\,(k\,d-t))]\,b'}{t\,(2\,k\,d-t)\,b + (k\,d-t)^2\,b'} \qquad (25a)$$

Arm of resisting couple (see footnote, Section 106),

$$j\,d = d - z \qquad (26a)$$

Compressive unit stress in extreme fiber of concrete,

$$f_c = \frac{2\,M\,k\,d}{[(2\,k\,d-t)\,b\,t + (k\,d-t)^2\,b']\,j\,d} \qquad (27a)$$

Tensile unit stress in longitudinal reinforcement,

$$f_s = \frac{M}{A_s j\, a} \qquad (28a)$$

† For illustration of certain symbols as applied to typical T-beams, see Fig. 3, Appendix I.

119.—*Isolated Beams.*—Isolated beams, in which the T-form is used only for the purpose of providing additional compression area, shall have a flange thickness not less than one-half (½) the width of the web and a total flange width not more than four (4) times the web thickness.

D.—Diagonal Tension and Shear

120.—*Notation.*—The symbols used in Formulas (29) to (35) are defined in Section 105, except as follows:

A_v = total area of web reinforcement in tension within a distance, s, that is, s_1, s_2, s_3 . . . . s_n, or the total area of all bars bent up in any one plane;

α = angle between web bars and longitudinal bars;

F = total tension in a bar;

f_c' = ultimate compressive strength of concrete at the age of 28 days, based on tests of 6 by 12-in. or 8 by 16-in. cylinders made and tested in accordance with the "Standard Methods of Making and Storing Specimens of Concrete in the Field." (Serial Designation: C31-21) of the American Society for Testing Materials (Appendix XIII) and the "Tentative Methods of Making Compression Tests of Concrete" (Serial Designation: C39-21T) of the American Society for Testing Materials (Appendix XII);

f_v = tensile unit stress in web reinforcement;

Q = ratio of the average to the maximum bond stress computed by Formula (34) within the distance, y;

Σo = sum of perimeters of bars in one set;

r = ratio of cross-sectional area of negative reinforcement which crosses entirely over the column capital of a flat slab or over the dropped panel, to the total cross-sectional area of the negative reinforcement in the two column strips;

s = spacing of web bars or stirrups measured at the plane of the lower reinforcement and in the direction of the longitudinal axis of the beam;

t_1 = thickness of flat slab without dropped panels or thickness of a dropped panel (see Fig. 14, Appendix I);

t_2 = thickness of flat slab with dropped panels at points away from the dropped panel (see Fig. 14, Appendix I);

u = bond stress per unit of area of surface of bar;

v = shearing unit stress;

V = total shear;

x = length of bar added for anchorage, including the hook, if any;

y = distance from the point at which tension is computed to the point of beginning of anchorage.

121.—*Formula for Shear.*—The shearing unit stress, v, in reinforced concrete beams shall be taken as not less than that computed by Formula (29).*

$$v = \frac{V}{b\,j\,d} \qquad (29)$$

* For I- or T-beams, b is the width of the stem as given in Section 123.

122.—*Variation of Shear in Beams with Uniform Load.*—For purpose of design of beams carrying uniform loads, not less than one-fourth ($\frac{1}{4}$) of the total shearing resistance required at either end of the span shall be provided at the section where the computed shearing stress is zero; from that section to the ends of the span the required shearing resistance shall be assumed to vary uniformly.

123.—*Width of Beams in Shear Computations.*—The shearing unit stress shall be computed on the minimum width of rectangular beams and on the minimum thickness of the web in beams of I- or T-section.

124.—*Shear in Beam and Tile Construction.*—The width of the effective section for shear as governing diagonal tension shall be assumed as the thickness of the concrete web plus one-half ($\frac{1}{2}$) the thickness of the vertical webs of the concrete or clay tile in contact with the beam. (For typical design, see Fig. 13, Appendix I).

125.—*Types and Spacing of Web Reinforcement.*—Web reinforcement may consist of:

(*a*) Vertical stirrups or web reinforcing bars.

(*b*) Inclined stirrups or web reinforcing bars forming an angle of 30° or more with the longitudinal bars.

(*c*) Longitudinal bars bent up at an angle of 15° or more with the direction of the longitudinal bars.

Stirrups or bent-up bars which are not anchored at both ends, according to the provisions of Section 141, shall not be considered effective as web reinforcement. When the shearing stress is not greater than $0.06f_c'$, the distance, s, measured in the direction of the axis of the beam between two successive stirrups, or between two successive points of bending up of bars, or from the point of bending up of a bar to the edge of the support, shall not be greater than

$$s = \frac{45\,d}{\alpha + 10} \qquad (30)$$

in which the angle, α, is in degrees.

When the shearing stress is greater than $0.06f_c'$, the distance, s, shall not be greater than two-thirds ($\frac{2}{3}$) of the values given by Formula (30).

126.—*Anchorage of Web Reinforcement.*—See Section 141.

127.—*Beams without Special Anchorage of Longitudinal Reinforcement.*—The shearing unit stress computed by Formula (29) in beams in which the longitudinal reinforcement is without special anchorage shall not exceed the values given by Formulas (31) and (32) and in no case shall it exceed $0.06f'_c$.

When α is between 45° and 90°,

$$v = 0.02\,f'_c + \frac{f_v\,A_v}{b\,s\,\sin\,\alpha} \qquad (31)$$

When α is less than 45°,

$$v = 0.02\,f'_c + \frac{f_v\,A_v}{b\,s}\,(\sin\,\alpha + \cos\,\alpha) \qquad (32)$$

128.—*Beams with Special Anchorage of Longitudinal Reinforcement.*—The shearing unit stress computed by Formula (29) in beams in which longitudinal reinforcement is anchored by means of hooked ends or otherwise, as specified in Section 140, shall not exceed the value given by Formulas (31) and (32), when $0.03f'_c$ is substituted for $0.02f'_c$ in those formulas; in no case shall the shearing unit stress exceed $0.12f'_c$.

129.—*Beams with Bars Bent Up at a Single Point.*—Where the web reinforcement consists of bars bent up at a single point, the point of bending shall be at a distance, s, from the edge of the support, not greater than that given in Section 125 and the value of the quantity, $\frac{f_v A_v}{b\,s}(\sin\alpha + \cos\alpha)$, used in the design shall not exceed 75 lb. per sq. in. (See Fig. 10, Appendix I.)

130.—*Combined Web Reinforcement.*—Where two or more types of web reinforcement are used in conjunction, the total shearing resistance of the beam shall be assumed as the sum of the shearing resistances computed for the various types separately. In such computations the shearing resistance of the concrete (the term, $0.02f'_c$ or $0.03f'_c$ in Formulas (31) and (32)) shall be included only once. In no case shall the maximum shearing stresses be greater than the limiting values given in Sections 127 and 128.

131.—*Shearing Stress in Flat Slabs.*—The shearing unit stress in flat slabs shall not exceed the value of v given by Formula (33):

$$v = 0.02f'_c\,(1 + r) \qquad (33)$$

and shall not in any case exceed $0.03f'_c$.

The shearing unit stress shall be computed on:

(a) A vertical section which has a depth, in inches, of $\frac{7}{8}(t_1 - 1\frac{1}{2})$ and which lies at a distance, in inches, of $t_1 - 1\frac{1}{2}$ from the edge of the column capital; and

(b) A vertical section which has a depth, in inches, of $\frac{7}{8}(t_2 - 1\frac{1}{2})$ and which lies at a distance, in inches, of $t_2 - 1\frac{1}{2}$ from the edge of the dropped panel.

In no case shall r be less than 0.25. Where the shearing stress computed as in Section (a) is being considered, r shall be assumed as the proportional amount of the negative reinforcement, within the column strip, crossing the column capital. Where the shearing stress computed as in Section (b) is being considered, r shall be assumed as the proportional amount of the negative reinforcement, within the column strip, crossing entirely over the dropped panel.* (For typical flat slab and designation of principal design sections, see Figs. 14 and 15, Appendix I.)

132.—*Shear and Diagonal Tension in Footings.*—The shearing stress shall be taken as not less than that computed by Formula (29). The stress on the critical section, shall not exceed $0.02f'_c$ for footings with straight reinforcement bars, nor $0.03f'_c$ for footings in which the reinforcement bars are anchored at both ends by adequate hooks or otherwise as specified in Section 140.

* In special cases, where supported by satisfactory engineering analysis, diagonal tension reinforcement may be used and increased shearing stresses allowed in accordance with Sections 127 to 130.

133.—*Critical Section for Soil Footings.*—The critical section for diagonal tension in footings on soil shall be computed on a vertical section through the perimeter of the lower base of a frustum of a cone or pyramid which has a base angle of 45°, and which has for its top the base of the column or pedestal and for its lower base the plane at the centroid of longitudinal reinforcement.

134.—*Critical Section for Pile Footings.*—The critical section for diagonal tension in footings on piles shall be computed on a vertical section at the inner edge of the first row of piles entirely outside a section midway between the face of the column or pedestal and the section described in Section 133 for soil footings, but in no case outside of the section described in Section 133. The critical section for piles not arranged in rows shall be taken midway between the face of the column and the perimeter of the base of the frustum described in Section 133.

E.—Bond and Anchorage

135.—*Bond Stresses by Beam Action.*—Where bar reinforcement is used to resist tensile stresses developed by beam action, the bond stress shall be taken as not less than that computed by Formula (34):

$$u = \frac{V}{\Sigma o j d} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (34)$$

For continuous or restrained members, the critical section for bond for the positive reinforcement shall be assumed to be at the point of inflection; that for the negative reinforcement shall be assumed to be at the face of the support, and at the point of inflection. For simple beams or freely supported end spans of continuous beams the critical section for bond shall be assumed to be at the face of the support.

Bent-up longitudinal bars which, at the critical section, are within a distance, $\frac{d}{3}$, from horizontal reinforcement under consideration, may be included with the straight bars in computing Σo.

In footings only the bars specified in Section 177 as effective in resisting bending moment shall be considered as resisting bond stresses. Special investigation shall be made of bond stresses in footings with stepped or sloping upper surface, as maximum bond stresses may occur at the vertical plane of the steps or near the edges of the footing.

136.—*Bond Stress for Ordinary Anchorage.*—In beams where the ordinary anchorage described in Section 139 is provided, the bond stress computed by Formula (34) at any section shall not exceed the following values:

For plain bars. . . . . $u = 0.04 f'_c$

For deformed bars meeting the requirements of Section 23. . . . $u = 0.05 f'_c$

137.—*Bond Stresses for Special Anchorage.*—In beams where special anchorage of the bars is provided as specified in Section 140, bond stresses exceeding those specified in Section 136 may be used, provided the total tensile stress at a point of abrupt change in stress or at the point of maximum stress, does not exceed the value of F given by Formula (35):

$$F = Q\, u\, \Sigma o\, y + u\, \Sigma o\, x \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (35)$$

in which,

F = total tension in the bar;

Σo = the perimeter of the bar under consideration;

Q = ratio of the average to the maximum bond stress computed by Formula (34) within the distance, y;

u = permissible bond stress = $0.04f'_c$ for plain and $0.05f'_c$ for deformed bars meeting the requirements of Section 23;

x = the length of bar added for anchorage, including the hook, if any;

y = distance from the point at which the tension is computed to the point of beginning of anchorage.

The length of bar added for anchorage may be either straight or bent. The radius of bend shall not be less than four (4) bar diameters.

138.—*Bond Stress for Reinforcement in Two or More Directions.*—The permissible bond stress for footings and similar members in which reinforcement is placed in more than one direction shall not exceed 75% of the values in Sections 136 and 137.

139.—*Ordinary Anchorage Requirements.*—In continuous, restrained or cantilever beams, anchorage of the tensile negative reinforcement beyond the face of the support shall provide for the full maximum tension with bond stresses not greater than those specified in Section 136. Such anchorage shall provide a length of bar not less than the depth of the beam. In the case of end supports which have a width less than three-fourths (¾) of the depth of the beam, the bars shall be bent down toward the support a distance not less than the effective depth of the beam. The portion of the bar so bent down shall be as near to the end of the beam as protective covering permits. (See Fig. 9, Appendix I.) In continuous or restrained beams, negative reinforcement shall be carried to or beyond the point of inflection. Not less than one-fourth (¼) of the area of the positive reinforcement shall extend into the support to provide an embedment of ten (10) or more bar diameters.

In simple beams, or freely supported end spans of continuous beams, at least one-fourth (¼) of the area of the tensile reinforcement shall extend along the tension side of the beam and beyond the face of the support to provide an embedment of ten (10) or more bar diameters.

140.—*Special Anchorage Requirements.*—Where increased shearing stresses are used as provided in Sections 128 and 132, or increased bond stresses as provided in Section 137 are necessary, special anchorage of all reinforcement in addition to that required in Section 139 shall be provided as follows:

(*a*) In continuous and restrained beams, anchorage beyond points of inflection of one-third (⅓) the area of the negative reinforcement and beyond the face of the support of one-third (⅓) the area of the positive reinforcement, shall be provided to develop one-third (⅓) of the maximum working stress in tension, with bond stresses not greater than those specified in Section 136.

(*b*) At the edges of footings, anchorage for all the bars for one-third (⅓) the maximum working stress in tension, shall be provided within a region where the tension in the concrete, computed as an unreinforced beam, does not exceed 40 lb. per sq. in.

(*c*) In simple beams, or freely-supported end spans of continuous beams, at least one-half ($\frac{1}{2}$) of the tensile reinforcement shall extend along the tension side of the beam to provide an anchorage beyond the face of the support for one-third ($\frac{1}{3}$) of the maximum working stress in tension.

141.—*Anchorage of Web Reinforcement.*—Web bars shall be anchored at both ends by:

(*a*) Providing continuity with the longitudinal reinforcement; or

(*b*) Bending around the longitudinal bar; or

(*c*) A semi-circular hook which has a radius not less than four (4) times the diameter of the web bar.

Stirrup anchorage shall be so provided in the compression and tension regions of a beam as to permit the development of safe working tensile stress in the stirrup at a point 0.3*d* from either face.*

The end anchorage of a web member not in bearing on the longitudinal reinforcement shall be such as to engage an amount of concrete sufficient to prevent the bar from pulling out. In all cases the stirrups shall be carried as close to the upper and lower surfaces as fireproofing requirements permit. (For typical designs see Figs. 8 and 12, Appendix I.)

F.—Flat Slabs

(Two-Way and Four-Way Systems with Rectangular Panels)

142.—*Moments in Interior Panels.*—The moment coefficients, moment distribution, and slab thicknesses specified herein are for slabs which have three (3) or more rows of panels in each direction, and in which the panels are approximately uniform in size. Slabs with paneled ceiling or with depressed paneling in the floor shall be considered as coming under the requirements herein given. The symbols used in Formulas (36) to (41) are defined in Section 105, except as indicated in Sections 142, 145, and 155.

In flat slabs in which the ratio of reinforcement for negative moment in the column strip is not greater than 0.01, the numerical sum of the positive and negative moments in the direction of either side of the panel for which tension reinforcement must be provided, shall be assumed as not less than that given by Formula (36):

$$M_0 = 0.09\ W\ l \left(1 - \frac{2\,c}{3\,l}\right)^2 \qquad (36)$$

in which,

M_0 = sum of positive and negative bending moments† in either rectangular direction, at the principal design sections of a panel of a flat slab;

* Generally, a properly anchored stirrup the diameter of which does not exceed one-fiftieth ($\frac{1}{50}$) of the depth of the beam will meet these requirements.

† The sum of the positive and negative moments provided for by this equation is about 72% of the moment found by rigid analysis based on the principles of mechanics. Extensive tests and experience with existing structures have shown that the requirements here stated will give adequate strength. See "Statical Limitations Upon the Steel Requirement in Reinforced Concrete Flat Slab Floors", by John R. Nichols, *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1670, and "Moments and Stresses in Slabs", by Messrs. H. M. Westergaard and W. A. Slater, *Proceedings*, Am. Concrete Inst., Vol. XVII (1921).

c = base diameter of the largest right circular cone which lies entirely within the column (including the capital) the vertex angle of which is 90° and the base of which is 1½ in. below the bottom of the slab or the bottom of the dropped panel (see Fig. 14, Appendix I);

l = span length* of flat slab, center to center of columns, in the rectangular direction in which moments are considered;

l_1 = span length* of flat slab, center to center of columns, perpendicular to the rectangular direction in which moments are considered; and

W = total dead and live load uniformly distributed over a single panel area.

143.—*Principal Design Sections.*—In computing the critical moments in flat slabs subjected to uniform load the following principal design sections shall be used:

(*a*) Section for Negative Moment in Middle Strip: The section beginning at a point on the edge of the panel, $\frac{l_1}{4}$, from the column center and extending in a rectangular direction a distance, $\frac{l_1}{2}$, toward the center of the adjacent column on the same panel edge (See Fig. 15, Appendix I).

(*b*) Section for Negative Moment in Column Strip: The section beginning at a point on the edge of the panel, $\frac{l_1}{4}$, from the center of a column and extending in a rectangular direction toward the column to a point, $\frac{c}{2}$, therefrom and thence along a one-quarter (¼) circumference about the column center to the adjacent edge of the panel.

(*c*) Section for Positive Moment in Middle Strip: The section of a length, $\frac{l_1}{2}$, extending in a rectangular direction across the center of the middle strip.

(*d*) Section for Positive Moment in Column Strip: The section of length, $\frac{l_1}{4}$, extending in a rectangular direction across the center of the column strip.

144.—*Moments in Principal Design Sections.*—The moments in the principal design sections shall be those given in Table 6, except as follows:

(*a*) The sum of the maximum negative moments in the two column strips may be greater or less than the values given in Table 6 by not more than $0.03\,M_0$.

* The column strip and the middle strip to be used when considering moments in the direction of the dimension, l, are located and dimensioned as shown in Fig. 15, Appendix I. The dimension, l_1, does not always represent the short length of the panel. When moments in the direction of the shorter panel length are considered, the dimensions, l and l_1, are to be interchanged and the strips corresponding to those shown in Fig. 15, but extending in the direction of the shorter panel length, are to be considered.

(*b*) The maximum negative moment and the maximum positive moment in the middle strip and the sum of the maximum positive moments in the two column strips may each be greater or less than the values given in Table 6 by not more than $0.01\,M_0$.

TABLE 6.—MOMENTS TO BE USED IN DESIGN OF FLAT SLABS.*

Strip.	FLAT SLABS WITHOUT DROPPED PANELS.		FLAT SLABS WITH DROPPED PANELS.	
	Negative.	Positive.	Negative.	Positive.
SLABS WITH TWO-WAY REINFORCEMENT.				
Column strip	$0.23\,M_0$	$0.11\,M_0$	$0.25\,M_0$	$0.10\,M_0$
Two column strips	$0.46\,M_0$	$0.22\,M_0$	$0.50\,M_0$	$0.20\,M_0$
Middle strip	$0.16\,M_0$	$0.16\,M_0$	$0.15\,M_0$	$0.15\,M_0$
SLABS WITH FOUR-WAY REINFORCEMENT.				
Column strip	$0.25\,M_0$	$0.10\,M_0$	$0.27\,M_0$	$0.095\,M_0$
Two column strips	$0.50\,M_0$	$0.20\,M_0$	$0.54\,M_0$	$0.190\,M_0$
Middle strip	$0.10\,M_0$	$0.20\,M_0$	$0.08\,M_0$	$0.190\,M_0$

* These are approximately the values which would be obtained by considering one-third ($\frac{1}{3}$) of the total moment, M_0, as positive and two-thirds ($\frac{2}{3}$) of it as negative moment.

145.—*Thickness of Flat Slabs and Dropped Panels.*—The total thickness,* t_1, of the dropped panel, in inches, or of the slab if a dropped panel is not used, shall be not less than:

$$t_1 = 0.038\left(1 - 1.44\,\frac{c}{l}\right) l \sqrt{R\,w'\,\frac{l_1}{b_1}} + 1\tfrac{1}{2} \quad \ldots\ldots\ldots (37)\dagger$$

in which,

R = ratio of negative moment in the two column strips to M_0;

w' = uniformly distributed dead and live load per unit of area of floor; and

b_1 = dimension of the dropped panel in the direction parallel to l_1.

For slabs with dropped panels the total thickness,* in inches, at points beyond the dropped panel shall be not less than:

$$t_2 = 0.02\,l\sqrt{w'} + 1 \quad \ldots\ldots\ldots (38)$$

The slab thickness, t_1 or t_2, shall in no case be less than $\frac{l}{32}$ for floor-slabs, and not less than $\frac{l}{40}$ for roof-slabs. In determining minimum thickness by Formulas (37) and (38), the value of l shall be the panel length, center to center of the columns, on the long side of the panel, l_1 shall be the panel length on the short side of the panel, and b_1 shall be the width or diameter of the

* The thickness will be in inches regardless of whether l and w' are in feet and pounds per square foot, or in inches and pounds per square inch.

† The values of R used in this formula are the coefficients of M_0 for negative moment in two column strips in Table 6.

dropped panel in the direction of l_1, except that in a slab without a dropped panel, b_1 shall be $0.5\,l_1$.

146.—*Minimum Dimensions of Dropped Panels.*—The dropped panel shall have a length or diameter in each rectangular direction of not less than one-third ($\frac{1}{3}$) the panel length in that direction, and a thickness not greater than $1.5\,t_2$.

147.—*Wall and Other Irregular Panels.*—In wall panels and other panels in which the slab is discontinuous at the edge of the panel, the maximum negative moment one panel length away from the discontinuous edge and the maximum positive moment between shall be increased as follows:

(*a*) Column strip perpendicular to the wall or discontinuous edge, 15% greater than that given in Table 6.

(*b*) Middle strip perpendicular to wall or discontinuous edge, 30% greater than that given in Table 6.

In these strips the bars used for positive moments perpendicular to the discontinuous edge shall extend to the edge of the panel at which the slab is discontinuous.

148.—*Panels with Marginal Beams.*—In panels having a marginal beam on one edge or on each of two adjacent edges, the beam shall be designed to carry at least the load superimposed directly upon it, exclusive of the panel load. A beam which has a depth greater than the thickness of the dropped panel into which it frames, shall be designed to carry, in addition to the load superimposed upon it, at least one-fourth ($\frac{1}{4}$) of the distributed load for which the adjacent panel or panels are designed, and each column strip adjacent to and parallel with the beam shall be designed to resist a moment at least one-half ($\frac{1}{2}$) as great as that specified in Table 6 for a column strip.*

Each column strip adjacent to and parallel with a marginal beam which has a depth less than the thickness of the dropped panel into which it frames shall be designed to resist the moments specified in Table 6 for a column strip. Marginal beams on opposite edges of a panel and the slab between them shall be designed for the entire load and the panels shall be designed as simple beams.

149.—*Discontinuous Panels.*—The negative moments on sections at and parallel to the wall, or discontinuous edge of an interior panel, shall be determined by the conditions of restraint.†

150.—*Flat Slabs on Bearing Walls.*—Where there is a beam or a bearing wall on the center line of columns in the interior portion of a continuous flat slab, the negative moment at the beam or wall line in the middle strip perpendicular to the beam or wall shall be taken as 30% greater than the moment specified in Table 6 for a middle strip. The column strip adjacent to and lying on either side of the beam or wall shall be designed to resist a moment at least one-half ($\frac{1}{2}$) of that specified in Table 6 for a column strip.

151.—*Point of Inflection.*—The point of inflection in any line parallel to a panel edge in interior panels of symmetrical slabs without dropped panels shall be assumed to be at a distance from the center of the span equal to 0.30

* In wall columns, brackets are sometimes substituted for capitals, or other changes are made in the design of the capital. Attention is directed to the necessity for taking into account the change in the value of c in the moment formula for such cases.

† The Committee is not prepared to make a more definite recommendation at this time.

of the distance between the two sections of critical negative moment at opposite ends of the line; for slabs having dropped panels, the coefficient shall be 0.25.

152.—*Reinforcement.*—The reinforcement bars which cross any section and which fulfill the requirements given in Section 153 may be considered as effective in resisting the moment at the section. The sectional area of a bar multiplied by the cosine of the angle between the direction of the axis of the bar and any other direction may be considered effective as reinforcement in that direction.

153.—*Arrangement of Reinforcement.*—The design shall include adequate provision for securing the reinforcement in place so as to take not only the critical moments, but the moments at intermediate sections. Provision shall be made for possible shifting of the point of inflection by carrying all bars in rectangular or diagonal directions, each side of a section of critical moment, either positive or negative, to points at least twenty (20) diameters beyond the point of inflection as specified in Section 151. Lapped splices shall not be permitted at or near regions of maximum stress except as described. At least four-tenths ($\frac{4}{10}$) of all bars in each direction shall be of such length and shall be so placed as to provide reinforcement at two sections of critical negative moment and at the intermediate section of critical positive moment. Not less than one-third ($\frac{1}{3}$) of the bars used for positive reinforcement in the column strip shall extend into the dropped panel not less than twenty (20) diameters of the bar, or in case no dropped panel is used, shall extend to a point not less than one-eighth ($\frac{1}{8}$) of the span length from the center line of the column or the support.

154.—*Reinforcement at Construction Joints.*—See Section 72.

155.—*Tensile Stress in Reinforcement.*—The tensile stress, f_s, in the reinforcement in flat slabs shall be taken as not less than that computed by Formula (39):

$$f_s = \frac{R\,M_0}{A_s\,j\,d} \qquad (39)$$

in which,

$R\,M_0$ = moment specified in Section 144 for two column strips or for one middle strip; and

A_s = effective cross-sectional area of the reinforcement which crosses any of the principal design sections and which meets the requirements of Section 153. The stress so computed shall not at any of the principal design sections exceed the values specified in Section 194.

156.—*Compressive Stress in Concrete.*—The compressive stress in the concrete in flat slabs shall be taken as not less than that computed by Formulas (40) and (41), but the stress so computed shall not exceed $0.4\,f'_c$.

Compression due to negative moment, $R\,M_0$, in the two column strips:

$$f_c = \frac{3.5\,R\,M_0}{0.67\,\sqrt[3]{p\,n\,b_1\,d^2}}\left(1 - 1.2\,\frac{c}{l}\right) \qquad (40)$$

in which, b_1 is as specified in Section 145.

Compression due to positive moment, $R M_0$, in the two column strips, or negative or positive moment in the middle strip:

$$f_c = \frac{6\ R\ M_0}{0.67 \sqrt[3]{p\ n}\ l_1\ d^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (41)$$

In special cases supported by satisfactory engineering analysis, approved by the Engineer, compression reinforcement may be used to increase the resistance to compression in accordance with other provisions of these specifications.

157.—*Shearing Stress.*—See Section 131.

158.—*Unusual Panels.*—For structures having a width of one or two panels, and also for slabs having panels of markedly different sizes, an analysis shall be made of the moments developed in both slab and columns, and the values given in Sections 142 to 157 modified accordingly.

159.—*Bending Moments in Columns.*—See Section 171.

G.—Reinforced Concrete Columns

160.—*Limiting Dimensions.*—The following sections on reinforced concrete columns are based on the assumption of a short column. Where the unsupported length is greater than forty (40) times the least radius of gyration (40 R), the safe load shall be determined by Formula (47). Principal columns in buildings shall have a minimum diameter or thickness of 12 in. Posts that are not continuous from story to story shall have a minimum diameter or thickness of 6 in.

161.—*Unsupported Length.*—The unsupported length of reinforced concrete columns shall be taken as:

(*a*) In flat slab construction the clear distance between the floor and under side of the capital.

(*b*) In beam-and-slab construction, the clear distance between the floor and the under side of the shallowest beam framing into the column at the next higher floor level.

(*c*) In floor construction with beams in one direction only, the clear distance between floor slabs.

(*d*) In columns supported laterally by struts or beams only, the clear distance between consecutive pairs (or groups) of struts or beams, provided that to be considered an adequate support, two such struts or beams shall meet the column at approximately the same level and the angle between the two planes formed by the axis of the column and the axis of each strut, respectively, is not less than 75 nor more than 105 degrees.

When haunches are used at the junction of beams or struts with columns, the clear distance between supports may be considered as reduced by two-thirds ($\frac{2}{3}$) of the depth of the haunch.

162.—*Safe Load on Spiral Columns.*—The safe axial load on columns reinforced with longitudinal bars and closely spaced spirals enclosing a circular core shall be not greater than that determined by Formula (42):*

$$P = A_c\ f_c + n\ f_c\ p\ A \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (42)$$

* The symbols used in Formulas (42) to (49) are defined in Section 105, except as indicated in Sections 162, 165, 168, 170, 176, and 182.

in which,

P = total safe axial load on column the $\frac{h}{R}$ of which is less than 40;

A = area of the concrete core enclosed within the spiral; the diameter of the core (or of the spiral) shall be taken as the distance center to center of the spiral wire;

p = ratio of effective area of longitudinal reinforcement to area of the concrete core;

$A_c = A\ (1 - p)$ = net area of concrete core;

f_c = permissible compressive stress in concrete =

$$300 + (0.10 + 4_p)\ f'_c \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (43)$$

The longitudinal reinforcement shall consist of at least six (6) bars of minimum diameter of ½ in. and its effective cross-sectional area shall not be less than 1%, nor more than 6% of the core.

163.—*Spiral Reinforcement.*—The spiral reinforcement shall be not less than one-fourth (¼) the volume of the longitudinal reinforcement; it shall consist of evenly spaced continuous spirals held firmly in place and true to line by at least three vertical spacer bars. The spacing of the spirals shall be not greater than one-sixth (⅙) of the diameter of the core and in no case more than 3 in. The spiral reinforcement shall meet the requirements of the "Tentative Specifications for Cold-Drawn Steel Wire for Concrete Reinforcement" (Serial Designation A82-21T) of the American Society for Testing Materials (Appendix VII).

164.—*Protection of Spirally Reinforced Column.*—Reinforcement shall be protected everywhere by a covering of concrete cast monolithic with the core, which shall have a minimum thickness of 1½ in. in square columns and 2 in. in round or octagonal columns.

165.—*Safe Load on Columns with Lateral Ties.*—The safe axial load on columns reinforced with longitudinal bars and separate lateral ties shall be not greater than that determined by Formula (44):

$$P = (A'_c + A_s n)\ f_c \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (44)$$

in which,

A'_c = net area of concrete in the column (total column area minus area of reinforcement);

A_s = effective cross-sectional area of longitudinal reinforcement;

f_c = (the permissible compressive stress in concrete) shall not exceed $0.20\ f'_c$.

The amount of longitudinal reinforcement considered in the calculations shall be not more than 2%, nor less than 0.5% of the total area of the column. The longitudinal reinforcement shall consist of not less than four (4) bars of minimum diameter of ½ in., placed with a clear distance from the face of the column of not less than 2 in.

166.—*Lateral Ties.*—Lateral ties shall be not less than ¼ in. in diameter, spaced not more than 8 in. apart.

167.—*Bending in Columns.*—Reinforced concrete columns subject to bending stresses shall be treated as follows:

(*a*) With spiral reinforcement: The compressive unit stress on the concrete within the core area under combined axial load and bending shall not exceed by more than 20% the value given for axial load by Formula (43).

(*b*) With lateral ties: Additional longitudinal reinforcement may be used if required, and the compressive unit stress on the concrete under combined axial load and bending may be increased to $0.30\,f'_c$. The total amount of reinforcement considered in the computations shall be not more than 4% of the total area of the column.

Tension in the longitudinal reinforcement due to bending of the column shall not exceed 16 000 lb. per sq. in.

168.—*Composite Columns.*—The safe load on composite columns in which a structural steel or cast-iron column is thoroughly encased in a circumferentially reinforced concrete core shall be based on a certain unit stress for the steel or cast-iron core plus a unit stress of $0.25\,f'_c$ on the area within the spiral core.

The unit compressive stress on the steel section shall be not greater than that determined by Formula (45):

$$f_r = 18\,000 - 70\,\frac{h}{R} \qquad (45)$$

but shall not exceed 16 000 lb. per sq. in.

The unit stress on the cast-iron section shall be not greater than that determined by Formula (46):

$$f_r = 12\,000 - 60\,\frac{h}{R} \qquad (46)$$

but shall not exceed 10 000 lb. per sq. in. In Formulas (45) and (46),

f_r = compressive unit stress in metal core; and
R = least radius of gyration of the steel or cast-iron section.

The diameter of the cast-iron section shall not exceed one-half (½) of the diameter of the core within the spiral. The spiral reinforcement shall be not less than 0.5% of the volume of the core within the spiral and shall conform in quality, spacing, and other requirements to the provisions for spirals in Section 163.

Ample section of concrete and continuity of reinforcement shall be provided at the junction with beams or girders. The area of the concrete between the spiral and the metal core shall be not less than that required to carry the total floor load of the story above on the basis of a stress in the concrete of $0.35\,f'_c$, unless special brackets are arranged on the metal core to receive directly the beam or slab load.

169.—*Structural Steel Columns.*—The safe load on a structural steel column of a section which fully encases an area of concrete and which is protected by an outside shell of concrete at least 3 in. thick, shall be computed in the same manner as for composite columns in Section 168, allowing $0.25\,f'_c$ on the area of the concrete enclosed by the steel section. The outside shell shall be reinforced by wire mesh, ties, or spiral hoops, weighing not less than 0.2 lb. per sq. ft. at the surface of the mesh, and with a maximum spacing of 6 in. between strands or hoops. Special brackets shall be used to receive the entire

floor load at each story. The safe load in steel columns calculated by Formula (45) shall not exceed 16 000 lb. per sq. in.

170.—*Long Columns.*—The permissible working load on the core in axially loaded columns which have a length greater than forty (40) times the least radius of gyration of the column core (40 R) shall be not greater than that determined by Formula (47):

$$\frac{P'}{P} = 1.33 - \frac{h}{120\,R} \qquad (47)$$

in which,

P' = total safe axial load on long column;

P = total safe axial load on column of the same section the $\frac{h}{R}$ of which is less than 40, determined as in Sections 162 and 165.

R = least radius of gyration of column core.

171.—*Bending Moments in Columns.*—The bending moments in interior and exterior columns shall be determined on the basis of loading conditions and end restraint, and shall be provided for in the design. The recognized methods shall be followed in calculating the stresses due to combined axial load and bending. In spiral columns, the area to be considered as resisting the stress is the area within the spiral.

H.—Footings

172.—*General.*—The requirements for tension, compression, shear, and bond in Sections 103 to 141, inclusive, shall govern the design of footings, except as hereinafter provided.

173.—*Soil Footings.*—The load per unit of area on soil footings shall be computed by dividing the column load by the area of base of the footing.

174.—*Pile Footings.*—Footings on piles shall be treated in the same manner as footings on soil, except that the load shall be considered as concentrated at the pile centers.

175.—*Sloped or Stepped Footings.*—Footings in which the thickness has been determined by the requirements for shear as specified in Sections 133 and 134 may be sloped or stepped between the critical section and the edge of the footing, provided that the shear on no section outside the critical section exceeds the value specified, and provided further that the thickness of the footing above the reinforcement at the edge shall not be less than 6 in. for footings on soil nor less than 12 in. for footings on piles. Sloped or stepped footings shall be cast as a unit.

176.—*Critical Section for Bending.*—The critical section for bending in a concrete footing which supports a concrete column or pedestal, shall be considered to be at the face of the column or pedestal. Where steel or cast-iron column bases are used, the moment in the footing shall be computed at the middle and at the edge of the base; the load shall be considered as uniformly distributed over the column or pedestal base.

The bending moment at the critical section in a square footing supporting a concentric square column, shall be computed from the load on the trapezoid

bounded by one face of the column, the corresponding outside edge of the footing, and the portions of the two diagonals. The load on the two corner triangles of this trapezoid shall be considered as applied at a distance from the face equal to six-tenths (0.6) of the projection of the footing from the face of the column. The load on the rectangular portion of the trapezoid shall be considered as applied at its center of gravity. The bending moment is expressed by Formula (48):

$$M = \frac{w}{2}\,(a + 1.2\,c)\,c^2 \qquad (48)$$

in which,

M = bending moment at critical section of footing;
a = width of face of column or pedestal;
c = projection of footing from face of column;
w = upward reaction per unit of area of base of footing.

For a round or octagonal column, the distance, a, shall be taken as equal to the side of a square of an area equal to the area enclosed within the perimeter of the column. (For typical footing designs, see Figs. 16 and 17, Appendix I.)

177.—*Reinforcement.*—The reinforcement in each direction in the footing shall be determined as for a reinforced concrete beam; the effective depth shall be the distance from the top of the footing to the plane of the reinforcement. The sectional area of reinforcement shall be distributed uniformly across the footing unless the width is greater than the side of the column or pedestal plus twice the effective depth of the footing, in which case the width over which the reinforcement is spread may be increased to include one-half ($\frac{1}{2}$) the remaining width of the footing. In order that no considerable area of the footing shall remain unreinforced, additional reinforcement shall be placed outside of the width specified, but such reinforcement shall not be considered as effective in resisting the calculated bending moment. For the extra reinforcement a spacing double that within the effective belt may be used.

178.—*Concrete Stress.*—The extreme fiber stress in compression in the concrete shall be kept within the limits specified in Section 189. The extreme fiber stress in sloped or stepped footings shall be based on the exact shape of the section for a width not greater than that assumed effective for reinforcement.

179.—*Irregular Footings.*—A rectangular or irregularly shaped footing shall be computed by dividing it into rectangles or trapezoids tributary to the sides of the column, using the distance to the center of gravity of the area as the moment arm of the upward forces. Outstanding portions of combined footings shall be treated in the same manner. Other portions of combined footings shall be designed as beams or slabs.

180.—*Shearing Stresses.*—See Sections 132 to 134.

181.—*Bond Stress.*—See Sections 135 to 141.

182.—*Transfer of Stress at Base of Column.*—The compressive stress in longitudinal reinforcement at the base of a column shall be transferred to

the pedestal or footing by either dowels or distributing bases. When dowels are used, there shall be at least one for each column bar, and the total sectional area of the dowels shall be not less than the sectional area of the longitudinal reinforcement in the column. The dowels shall extend into the column and into the pedestal or footing not less than fifty (50) diameters of the dowel bars for plain bars, or forty (40) diameters for deformed bars.

When metal distributing bases are used, they shall have sufficient area and thickness to transmit safely the load from the longitudinal reinforcement in compression and bending. The permissible compressive unit stress on top of the pedestal or footing directly under the column shall be not greater than that determined by Formula (49):

$$r_a = 0.25\, f_c' \sqrt[3]{\frac{A}{A'}} \qquad (49)$$

in which,

r_a = permissible working stress over the loaded area;
A = total area at the top of the pedestal or footing;
A' = loaded area at the column base;
f_c' = ultimate compressive strength of concrete. (See Section 120.)

In sloped or stepped footings, A may be taken as the area of the top horizontal surface of the footing or as the area of the lower base of the largest frustrum of a pyramid or cone contained wholly within the footing and having for its upper base the loaded area, A', and having side slopes of 1 vertical to 2 horizontal.

183.—*Pedestals Without Reinforcement.*—The allowable compressive unit stress on the gross area of a concentrically loaded pedestal or on the minimum area of a pedestal footing shall not exceed 0.25 f_c', unless reinforcement is provided and the member designed as a reinforced concrete column.

The depth of a pedestal or pedestal footing shall be not greater than three (3) times its least width and the projection on any side from the face of the supported member shall be not greater than one-half (½) the depth. The depth of a pedestal the sides of which are sloped or stepped shall not exceed three (3) times the least width or diameter of the section midway between the top and bottom. A pedestal footing supported directly on piles shall have a mat of reinforcing bars having a cross-sectional area of not less than 0.20 sq. in. per ft. in each direction, placed 3 in. above the top of the piles.

I.—Reinforced Concrete Retaining Walls

184.—*Loads and Unit Stresses.*—Reinforced concrete retaining walls shall be so designed* that the permissible unit stresses specified in Sections 186 to 197 are not exceeded. The heels of cantilever, counterforted, and buttressed retaining walls shall be proportioned for maximum resultant vertical loads, but when the foundation reaction is neglected, the permissible unit stresses shall not be more than 50% greater than the normal permissible stresses.

185.—*Details of Design.*—The following principles shall be followed in the design of reinforced concrete retaining walls:

* In proportioning retaining walls consideration should be given to: (*a*) Bearing value of soil; and (*b*) stability against sliding.

(*a*) The unsupported toe and heel of the base slabs shall be considered as cantilever beams fixed at the edge of the support.

(*b*) The vertical section of a cantilever wall shall be considered as a cantilever beam fixed at the top of the base.

(*c*) The vertical sections of counterforted and buttressed walls and parts of base slabs supported by the counterforts or buttresses shall be designed in accordance with the requirements for a continuous slab in Section 110.

(*d*) The exposed faces of walls without buttresses shall preferably be given a batter of not less than $\frac{1}{4}$ in. per ft.

(*e*) Counterforts shall be designed in accordance with the requirements for T-beams in Section 113 to 115. Stirrups shall be provided in the counterforts to take the reaction when the tension reinforcement of the face walls and heels of bases is designed to span between the counterforts. Stirrups shall be anchored as near the exposed face of the longitudinal wall and as close to the lower face of the base as the requirements for protective covering permit.

(*f*) Buttresses shall be designed in accordance with the requirements specified for rectangular beams.

(*g*) The shearing stress at the junction of the base with counterforts or buttresses shall not exceed the values specified in Sections 120 to 130.

(*h*) Horizontal metal reinforcement shall be of such form and distributed so as to develop the required bond. To prevent temperature and shrinkage cracks in exposed surface not less than 0.25 sq. in. of horizontal metal reinforcement per foot of height shall be provided.

(*i*) Grooved lock-joints shall be placed not more than 60 ft. apart to care for temperature changes.

(*j*) Counterforts and buttresses shall be located under all points of concentrated loading, and at intermediate points as may be required by the design.

(*k*) The walls shall be cast as a unit between expansion joints unless construction joints formed in accordance with Sections 69 and 73 are provided.

(*l*) Drains or "weep-holes" not less than 4 in. in diameter and not more than 10 ft. apart, shall be provided; at least one drain shall be provided for each pocket formed by counterforts.

J.—Summary of Working Stresses

186.—*General.*—The following notation shall be used, in which,

f'_c = ultimate compressive strength of concrete at age of 28 days, based on tests of 6 by 12-in. or 8 by 16-in. cylinders made and tested in accordance with the "Standard Methods of Making and Storing Specimens of Concrete in the Field" (Serial Designation: C31-21) of the American Society for Testing Materials (Appendix XIII) and the "Tentative Methods of Making Compression Tests of Concrete" (Serial Designation: C39-21T) of the American Society for Testing Materials (Appendix XII).

Direct Stress in Concrete

187.—*Direct Compression.*—

(*a*) Columns the length of which does not exceed $40R$:

1.—With spirals...varies with amount of longitudinal reinforcement .....(See Section 162.)

2.—Longitudinal reinforcement and lateral ties..(See Section 165.)

(*b*) Long columns .....(See Section 170.)

(*c*) Piers and pedestals.....$0.25f'_c$ (See Section 183.)

188.—*Compression in Extreme Fiber.*—

(*a*) Extreme fiber stress in flexure.....$0.40f'_c$

(*b*) Extreme fiber stress in flexure adjacent to supports of continuous beams .....$0.45f'_c$

189.—*Tension.*—

In concrete members.....None

Shearing Stresses in Concrete

190.—*Longitudinal Bars without Special Anchorage.*—

(*a*) Beams without web reinforcement.....$0.02f'_c$

(*b*) Beams with stirrups or bent-up bars or combination of the two..$0.06f'_c$

191.—*Longitudinal Bars Having Special Anchorage.*—

(*a*) Beams without web reinforcement.....$0.02f'_c$

(*b*) Beams with stirrups or bent-up bars or a combination of the two.$0.12f'_c$

192.—*Flat Slabs.*—

(*a*) Shear at distance, *d*, from capital or dropped panel.....$0.03f'_c$

(*b*) Other limiting cases in flat slabs.....(See Section 131.)

193.—*Footings.*—

(*a*) Longitudinal bars without special anchorage.....$0.02f'_c$

(*b*) Longitudinal bars having special anchorage.....$0.03f'_c$

Stresses in Reinforcement

194.—*Tension in Steel.*—

(*a*) Billet-steel bars:

1.—Structural steel grade..... 16 000 lb. per sq. in.

2.—Intermediate grade ..... 18 000 " " " "

3.—Hard grade ..... 18 000 " " " "

(*b*) Rail-steel bars ..... 18 000 " " " "

(*c*) Structural steel ..... 16 000 " " " "

(*d*) Cold-drawn steel wire:

1.—Spirals ..... Stress not calculated

2.—Elsewhere ..... 18 000 lb. per sq. in.

195.—*Compression in Steel.*—

(*a*) Bars ..... Same as Section 194 (*a*) and (*b*)

(*b*) Structural steel core of composite column..... 16 000 lb. per sq. in.
Reduced for slenderness ratio..... (See Section 168.)

(*c*) Structural steel column..... 16 000 lb. per sq. in.
Reduced for slenderness ratio..... (See Section 169.)

196.—*Compression in Cast Iron.*—

Composite cast-iron column..... 10 000 lb. per sq. in.
Reduced for slenderness ratio..... (See Section 168.)

197.—*Bond Between Concrete and Reinforcement.*—

(*a*) Beams and slabs, plain bars..... $0.04f'_c$

(*b*) Beams and slabs, deformed bars..... $0.05f'_c$

(*c*) Footings, plain bars, one-way..... $0.04f'_c$

(*d*) Footings, deformed bars, one-way..... $0.05f'_c$

(*e*) Footings, bars, two-ways.... Sections (*c*) or (*d*) reduced by 25 per cent.

MANDATORY APPENDICES*

APPENDIX I

NOTATION AND DIAGRAMS

All symbols used in the Standard Specifications for Concrete and Reinforced Concrete have been collected here for convenience of reference. The symbols are in general defined in the text near the formulas in which they are used. In a few instances the same symbol is used in two distinct senses; however, there is little danger of confusion from this source.

NOTATION

a = width of face of column or pedestal;

α = angle between inclined web bars and longitudinal bars;

A = total net area of column, footing, or pedestal, exclusive of fireproofing;

A' = loaded area of pedestal, pier, or footing;

$A_c = A\,(1-p)$ = net area of concrete core of column (core area minus area of reinforcement);

A'_c = net area of concrete in columns with lateral ties (total column area minus area of reinforcement);

A_s = effective cross-sectional area of metal reinforcement in tension in beams or compression in columns; and the effective cross-sectional area of metal reinforcement which crosses any of the principal design sections of a flat slab and which meets the requirements of Section 153;

A_v = total area of web reinforcement in tension within a distance of s (s_1, s_2, s_3, etc.), or the total area of all bars bent up in any one plane (see Fig 9);

b = width of rectangular beam or width of flange of T-beam;

b' = width of stem of T-beam;

b_1 = dimension of the dropped panel of a flat slab in the direction parallel to l_1;†

c = base diameter of the largest right circular cone which lies entirely within the column (including the capital) the vertex angle of which is 90° and the base of which is 1½ in. below the bottom of the slab or the bottom of the dropped panel (see Fig. 14);

c = projection of footing from face of column;

C = total compressive stress in concrete;

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† In flat-slab design, the column strip, and the middle strip to be used when considering moments in the direction of the dimension, l, are located and dimensioned as shown in Fig. 15. The dimension, l_1, does not always represent the short length of the panel. When moments in the direction of the shorter panel length are considered, the dimensions, l and l_1, are to be interchanged and strips corresponding to those shown in Fig. 15, but extending in the direction of the shorter panel length, are to be considered.

C' = total compressive stress in reinforcement;

d = depth from compression surface of beam or slab to center of longitudinal tension reinforcement;

d' = depth from compression surface of beam or slab to center of compression reinforcement;

E_c = modulus of elasticity of concrete in compression;

E_s = modulus of elasticity of steel in tension = 30 000 000 lb. per sq. in.;

f_c = compressive unit stress in extreme fiber of concrete;

f'_c = ultimate compressive strength of concrete at age of 28 days, based on tests of 6 by 12-in. or 8 by 16-in. cylinders made and tested in accordance with the "Standard Methods of Making and Storing Specimens of Concrete in the Field" (Appendix XIII) and the "Tentative Methods of Making Compression Tests of Concrete" (Appendix XII);

f_r = compressive unit stress in metal core;

f_s = tensile unit stress in longitudinal reinforcement;

f'_s = compressive unit stress in longitudinal reinforcement;

f_v = tensile unit stress in web reinforcement;

F = total tension in a bar;

h = unsupported length of column;

I = moment of inertia of a section about the neutral axis for bending;

j = ratio of lever arm of resisting couple to depth, d;

$jd = d - z$ = arm of resisting couple;

k = ratio of depth of neutral axis to depth, d;

l = span length of beam or slab (generally distance from center to center of supports; for special cases, see Sections 106 and 145);

l = span length of flat slab, center to center of columns; in the rectangular direction in which moments are considered;*

l_1 = span length of flat slab, center to center of columns, perpendicular to the rectangular direction in which moments are considered;*

M = bending moment or moment of resistance in general;

M_0 = sum of positive and negative bending moments in either rectangular direction, at the principal design sections of a panel of a flat slab;

$n = \frac{E_s}{E_c}$ = ratio of modulus of elasticity of steel to that of concrete;

Σo = sum of perimeters of bars in one set;

p = ratio of effective area of tension reinforcement to effective area of concrete in beams = $\frac{A_s}{b\,d}$; and the ratio of effective area of

* See footnote, regarding b_1, in foregoing notation, p. 1212.

longitudinal reinforcement to the area of the concrete core in columns;

p' = ratio of effective area of compression reinforcement to effective area of concrete in beams;

P = total safe axial load on column the $\frac{h}{R}$ of which is less than 40;

P' = total safe axial load on long column;

Q = ratio of the average to the maximum bond stress computed by Formula (34) within the distance, y;

r = ratio of cross-sectional area of negative reinforcement which crosses entirely over the column capital of a flat slab or over the dropped panel, to the total cross-sectional area of the negative reinforcement in the two column strips;

r_a = permissible working stress in concrete over the loaded area of a pedestal, pier, or footing;

R = ratio of positive or negative moment in two column strips or one middle strip of a flat slab, to M_0;

R = least radius of gyration of a section;

s = spacing of web members, measured at the plane of the lower reinforcement and in the direction of the longitudinal axis of the beam;

t = thickness of flange of T-beam;

t_1 = thickness of flat slab without dropped panels or thickness of a dropped panel (see Fig. 14);

t_2 = thickness of flat slab with dropped panels at points away from the dropped panel (see Fig. 14);

T = total tensile stress in longitudinal reinforcement;

u = bond stress per unit of area of surface of bar;

v = shearing unit stress;

V = total shear;

w = uniformly distributed load per unit of length of beam or slab;

w = upward reaction per unit of area of base of footing;

w' = uniformly distributed dead and live load per unit of area of a floor or roof;

W = total dead and live load uniformly distributed over a single panel area;

x = length of bar added for anchorage, including the hook, if any;

y = distance from the point at which the tension is computed to the point of beginning of anchorage;

z = depth from compression surface of beam or slab to resultant of compressive stresses.

Diagrams

For explanation of symbols used in diagrams, see the foregoing Notation.

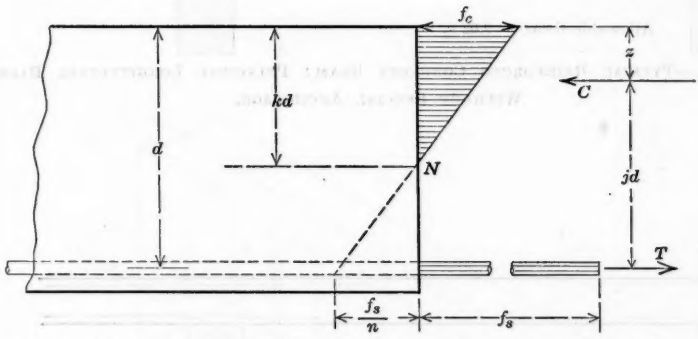


Fig. 1.—Nomeclature for Concrete Beam Reinforced for Tension.

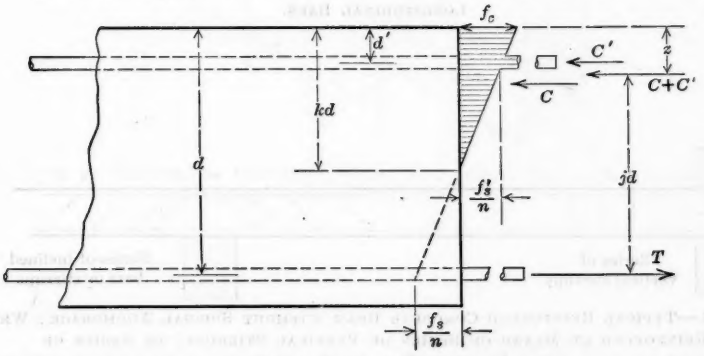


Fig. 2.—Nomenclature for Concrete Beam Reinforced for Tension and Compression.

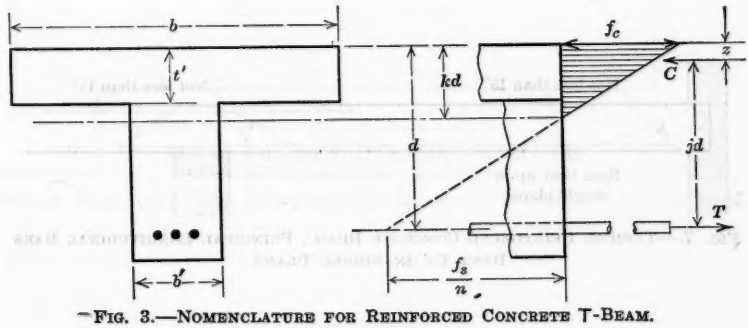


Fig. 3.—Nomenclature for Reinforced Concrete T-Beam.

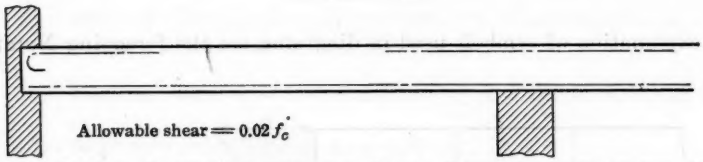


FIG. 4.—TYPICAL REINFORCED CONCRETE BEAM; PRINCIPAL LONGITUDINAL BARS WITHOUT SPECIAL ANCHORAGE.

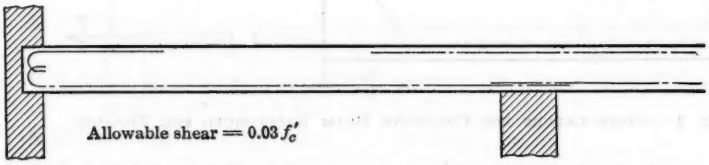


FIG. 5.—TYPICAL REINFORCED CONCRETE BEAM; SPECIAL ANCHORAGE OF LONGITUDINAL BARS.

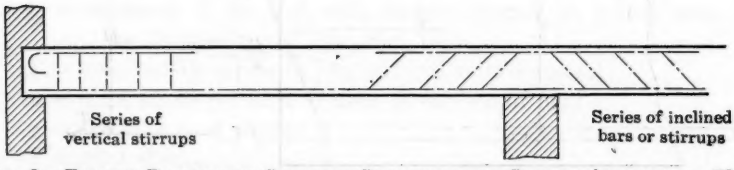


FIG. 6.—TYPICAL REINFORCED CONCRETE BEAM WITHOUT SPECIAL ANCHORAGE; WEB REINFORCED BY MEANS OF SERIES OF VERTICAL STIRRUPS; OR SERIES OF INCLINED BARS OR STIRRUPS.

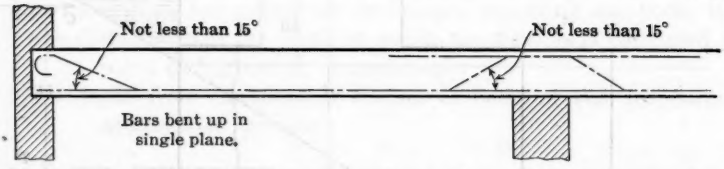


FIG. 7.—TYPICAL REINFORCED CONCRETE BEAM; PRINCIPAL LONGITUDINAL BARS BENT UP IN SINGLE PLANE.

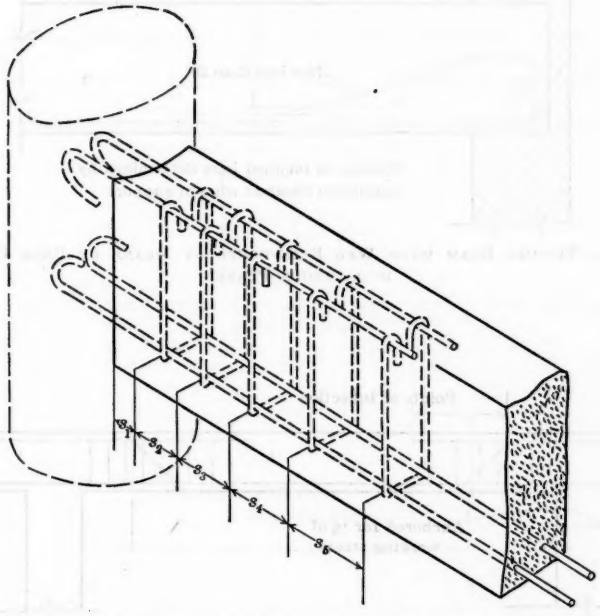


FIG. 8.—TYPICAL REINFORCED CONCRETE BEAM WITH ANCHORED LONGITUDINAL BARS AND VERTICAL STIRRUPS.

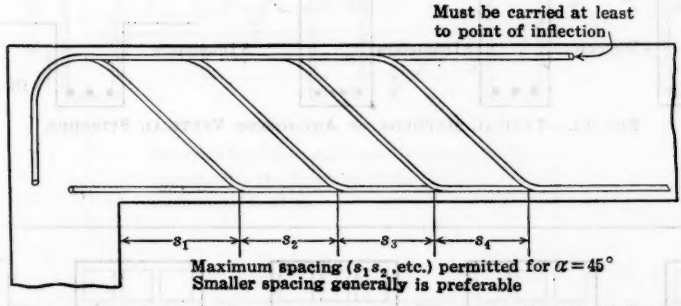

Must be carried at least to point of inflection
s_1
s_2
s_3
s_4
Maximum spacing ($s_1 s_2$, etc.) permitted for $\alpha = 45°$
Smaller spacing generally is preferable

FIG. 9.—TYPICAL BEAM WITH WEB REINFORCED BY MEANS OF SERIES OF INCLINED BARS.

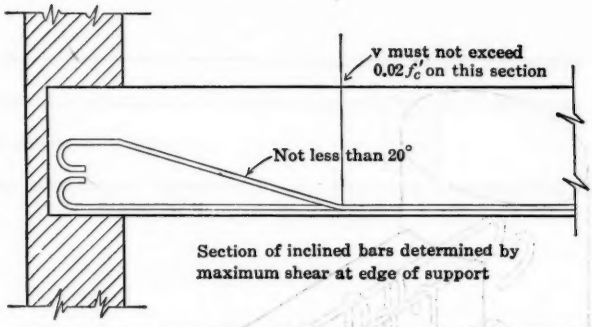

v must not exceed
0.02 f'_c on this section
Not less than 20°
Section of inclined bars determined by
maximum shear at edge of support

FIG. 10.—TYPICAL BEAM WITH WEB REINFORCED BY MEANS OF BARS BENT UP IN A SINGLE PLANE.

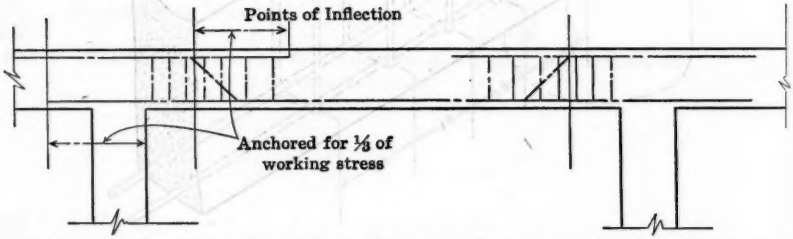


FIG. 11.—TYPICAL WEB REINFORCEMENT FOR CONTINUOUS BEAMS WITH SPECIAL ANCHORAGE.

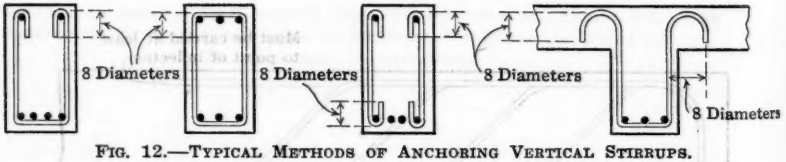


FIG. 12.—TYPICAL METHODS OF ANCHORING VERTICAL STIRRUPS.

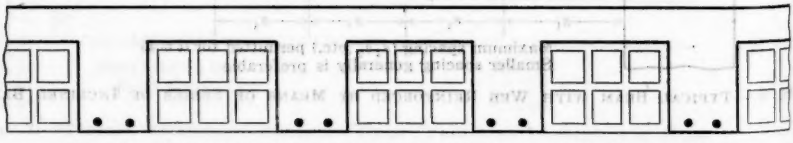


FIG. 13.—TYPICAL REINFORCED CONCRETE BEAM-AND-TILE CONSTRUCTION.

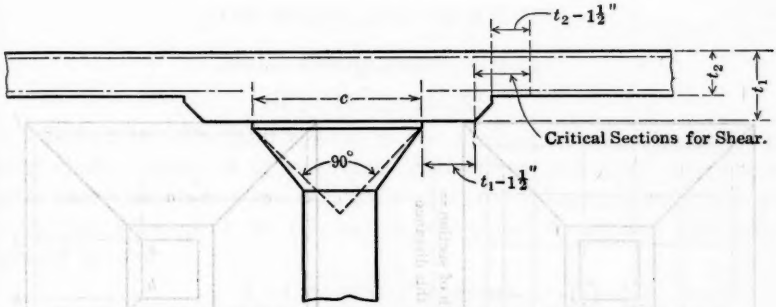


FIG. 14.—TYPICAL COLUMN CAPITAL AND SECTIONS OF FLAT SLAB WITH DROPPED PANEL.

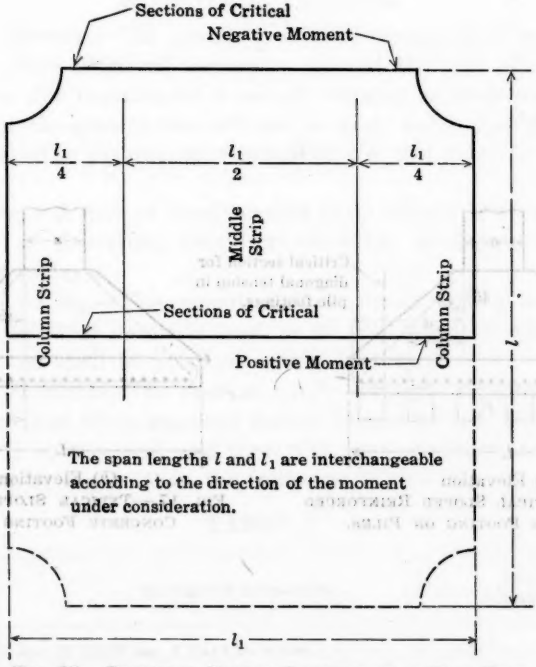
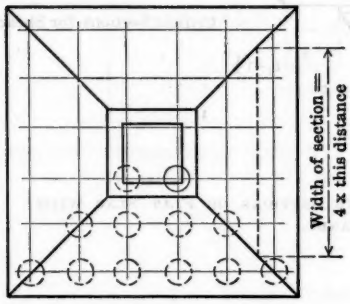
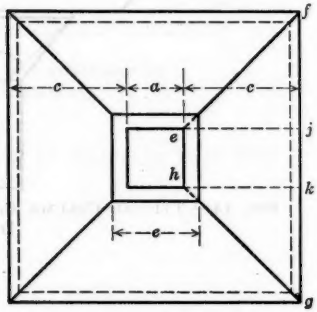


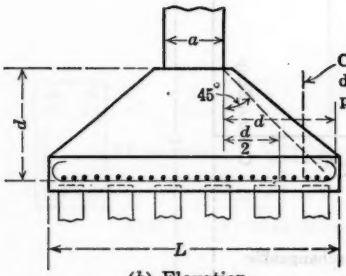
FIG. 15.—PRINCIPAL DESIGN SECTIONS OF A FLAT SLAB.



(*a*) Plan

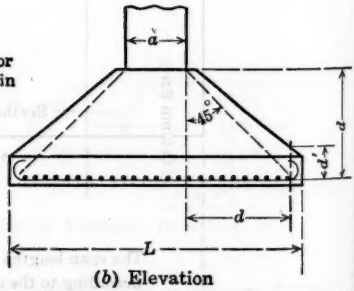


(*a*) Plan



(*b*) Elevation

FIG. 16.—TYPICAL SLOPED REINFORCED CONCRETE FOOTING ON PILES.



(*b*) Elevation

FIG. 17.—TYPICAL SLOPED REINFORCED CONCRETE FOOTING ON SOIL.

APPENDIX II

STANDARD SPECIFICATIONS AND TESTS FOR PORTLAND CEMENT*

Serial Designation: C 9-21

SPECIFICATIONS

1.—*Definition.*—Portland cement is the product obtained by finely pulverizing clinker produced by calcining to incipient fusion an intimate and properly proportioned mixture of argillaceous and calcareous materials, with no additions subsequent to calcination excepting water and calcined or uncalcined gypsum.

I.—Chemical Properties

2.—*Chemical Limits.*—The following limits shall not be exceeded:

Loss on ignition, per cent	4.00
Insoluble residue, per cent	0.85
Sulphuric anhydride (SO_3), per cent	2.00
Magnesia (MgO), per cent	5.00

II.—Physical Properties

3.—*Specific Gravity.*—The specific gravity of cement shall be not less than 3.10 (3.07 for white Portland cement). Should the test of cement as received fall below this requirement a second test may be made upon an ignited sample. The specific gravity test will not be made unless specifically ordered.

4.—*Fineness.*—The residue on a standard No. 200 sieve shall not exceed 22% by weight.

5.—*Soundness.*—A pat of neat cement shall remain firm and hard, and show no signs of distortion, cracking, checking, or disintegration in the steam test for soundness.

6.—*Time of Setting.*—The cement shall not develop initial set in less than 45 min. when the Vicat needle is used, or 60 min. when the Gillmore needle is used. Final set shall be attained within 10 hours.

7.—*Tensile Strength.*—The average tensile strength, in pounds per square inch, of not less than three standard mortar briquettes (see Section 50) composed of one (1) part cement and three (3) parts standard sand, by weight, shall be equal to or higher than the following (Table 7):

TABLE 7.

Age at test, days.	Storage of briquettes.	Tensile strength, in pounds per square inch.
7	1 day in moist air, 6 days in water	200
28	1 day in moist air, 27 days in water	300

* These specifications and tests are issued by the American Society for Testing Materials, under the fixed designation C 9; the final number indicates the year of original adoption as standard, or in the case of revision, the year of last revision. Adopted, 1904; revised, 1908, 1909, 1916, 1920 (effective January 1, 1921). These specifications were approved March 31, 1922, as "American Standard" by the American Engineering Standards Committee.

8.—The average tensile strength of standard mortar at 28 days shall be higher than the strength at 7 days.

III.—Packages, Marking, and Storage

9.—*Packages and Marking.*—The cement shall be delivered in suitable bags or barrels with the brand and name of the manufacturer plainly marked thereon, unless shipped in bulk. A bag shall contain 94 lb. net. A barrel shall contain 376 lb. net.

10.—*Storage.*—The cement shall be stored in such a manner as to permit easy access for proper inspection and identification of each shipment, and in a suitable weather-tight building which will protect the cement from dampness.

IV.—Inspection

11.—Every facility shall be provided the purchaser for careful sampling and inspection at either the mill or at the site of the work, as may be specified by the purchaser. At least 10 days from the time of sampling shall be allowed for the completion of the 7-day test, and at least 31 days shall be allowed for the completion of the 28-day test. The cement shall be tested in accordance with the methods hereinafter prescribed. The 28-day test shall be waived only when specifically so ordered.

V.—Rejection

12.—The cement may be rejected if it fails to meet any of the requirements of these specifications.

13.—Cement shall not be rejected on account of failure to meet the fineness requirement if upon re-test after drying at 100° cent. for 1 hour it meets this requirement.

14.—Cement failing to meet the test for soundness in steam may be accepted if it passes a re-test using a new sample at any time within 28 days thereafter.

15.—Packages varying more than 5% from the specified weight may be rejected; and if the average weight of packages in any shipment, as shown by weighing 50 packages taken at random, is less than that specified, the entire shipment may be rejected.

TESTS

VI.—Sampling

16.—*Number of Samples.*—Tests may be made on individual or composite samples as may be ordered. Each test sample should weigh at least 8 lb.

17.—(*a*).—*Individual Sample.*—If sampled in cars one test sample shall be taken from each 50 bbl. or fraction thereof. If sampled in bins one sample shall be taken from each 100 bbl.

(*b*).—*Composite Sample.*—If sampled in cars one sample shall be taken from 1 sack in each 40 sacks (or 1 bbl. in each 10 bbl.) and combined to form one test sample. If sampled in bins or warehouses one test sample shall represent not more than 200 bbl.

18.—*Method of Sampling.*—Cement may be sampled at the mill by any of the following methods that may be practicable, as ordered:

(*a*).—*From the Conveyor Delivering to the Bin.*—At least 8 lb. of cement shall be taken from approximately each 100 bbl. passing over the conveyor.

(*b*).—*From Filled Bins by Means of Proper Sampling Tubes.*—Tubes inserted vertically may be used for sampling cement to a maximum depth of 10 ft. Tubes inserted horizontally may be used where the construction of the bin permits. Samples shall be taken from points well distributed over the face of the bin.

(*c*).—*From Filled Bins at Points of Discharge.*—Sufficient cement shall be drawn from the discharge openings to obtain samples representative of the cement contained in the bin, as determined by the appearance at the discharge openings of indicators placed on the surface of the cement directly above these openings before drawing of the cement is started.

19.—*Treatment of Sample.*—Samples preferably shall be shipped and stored in air-tight containers. Samples shall be passed through a sieve having 20 meshes per linear inch in order thoroughly to mix the sample, break up lumps, and remove foreign materials.

VII.—Chemical Analysis

Loss on Ignition

20.—*Method.*—One gramme of cement shall be heated in a weighed covered platinum crucible, of 20 to 25 cu. cm. capacity, as follows, using either Method (*a*) or (*b*) as ordered:

(*a*) The crucible shall be placed in a hole in an asbestos board, clamped horizontally so that about three-fifths of the crucible projects below, and blasted at a full red heat for 15 min. with an inclined flame; the loss in weight shall be checked by a second blasting for 5 min. Care shall be taken to wipe off particles of asbestos that may adhere to the crucible when withdrawn from the hole in the board. Greater neatness and shortening of the time of heating are secured by making a hole to fit the crucible in a circular disk of sheet platinum and placing this disk over a somewhat larger hole in an asbestos board.

(*b*) The crucible shall be placed in a muffle at any temperature between 900 and 1 000° cent. for 15 min. and the loss in weight shall be checked by a second heating for 5 min.

21.—*Permissible Variation.*—A permissible variation of 0.25 will be allowed, and all results in excess of the specified limit but within this permissible variation shall be reported as 4 per cent.

Insoluble Residue

22.—*Method.*—To a 1-gramme sample of cement shall be added 10 cu. cm. of water and 5 cu. cm. of concentrated hydrochloric acid; the liquid shall be warmed until effervescence ceases. The solution shall be diluted to 50 cu. cm. and digested on a steam bath or hot plate until it is evident that decomposition of the cement is complete. The residue shall be filtered,

washed with cold water, and the filter paper and contents digested in about 30 cu. cm. of a 5% solution of sodium carbonate, the liquid being held at a temperature just short of boiling for 15 min. The remaining residue shall be filtered, washed with cold water, then with a few drops of hot hydrochloric acid, 1 : 9, and finally with hot water, and then ignited at a red heat and weighed as the insoluble residue.

23.—*Permissible Variation.*—A permissible variation of 0.15 will be allowed, and all results in excess of the specified limit but within this permissible variation shall be reported as 0.85 per cent.

Sulphuric Anhydride

24.—*Method.*—One gramme of the cement shall be dissolved in 5 cu. cm. of concentrated hydrochloric acid diluted with 5 cu. cm. of water, with gentle warming; when the solution is complete 40 cu. cm. of water shall be added, the solution filtered, and the residue washed thoroughly with water. The solution shall be diluted to 250 cu. cm., heated to boiling and 10 cu. cm. of a hot 10% solution of barium chloride shall be added slowly, drop by drop, from a pipette and the boiling continued until the precipitate is well formed. The solution shall be digested on the steam bath until the precipitate has settled. The precipitate shall be filtered, washed, and the paper and contents placed in a weighed platinum crucible and the paper slowly charred and consumed without flaming. The barium sulphate shall then be ignited and weighed. The weight obtained multiplied by 34.3 gives the percentage of sulphuric anhydride. The acid filtrate obtained in the determination of the insoluble residue may be used for the estimation of sulphuric anhydride instead of using a separate sample.

25.—*Permissible Variation.*—A permissible variation of 0.10 will be allowed, and all results in excess of the specified limit but within this permissible variation shall be reported as 2.00 per cent.

Magnesia

26.—*Method.*—To 0.5 gramme of the cement in an evaporating dish shall be added 10 cu. cm. of water to prevent lumping and then 10 cu. cm. of concentrated hydrochloric acid. The liquid shall be gently heated and agitated until the attack is complete. The solution shall then be evaporated to complete dryness on a steam or water bath. To hasten dehydration the residue may be heated to 150 or even 200° cent. for ½ to 1 hour. The residue shall be treated with 10 cu. cm. of concentrated hydrochloric acid diluted with an equal amount of water. The dish shall be covered and the solution digested for 10 min. on a steam bath or water bath. The diluted solution shall be filtered and the separated silica washed thoroughly with water.* Five cubic centimeters of concentrated hydrochloric acid and sufficient bromine water to precipitate any manganese which may be present, shall be added to the filtrate (about 250 cu. cm.). This shall be made alkaline with ammonium hydroxide, boiled until there is but a faint odor of ammonia,

* Since this procedure does not involve the determination of silica, a second evaporation is unnecessary.

and the precipitated iron and aluminum hydroxides, after settling, shall be washed with hot water, once by decantation and slightly on the filter. Setting aside the filtrate, the precipitate shall be transferred by a jet of hot water to the precipitating vessel and dissolved in 10 cu. cm. of hot hydrochloric acid. The paper shall be extracted with acid, the solution and washings being added to the main solution. The aluminum and iron shall then be reprecipitated at boiling heat by ammonium hydroxide and bromine water in a volume of about 100 cu. cm., and the second precipitate shall be collected and washed on the filter used in the first instance if this is still intact. To the combined filtrates from the hydroxides of iron and aluminum, reduced in volume if need be, 1 cu. cm. of ammonium hydroxide shall be added, the solution brought to boiling, 25 cu. cm. of a saturated solution of boiling ammonium oxalate added, and the boiling continued until the precipitated calcium oxalate has assumed a well defined granular form. The precipitate after 1 hour shall be filtered and washed, then with the filter shall be placed wet in a platinum crucible, and the paper burned off over a small flame of a Bunsen burner; after ignition it shall be re-dissolved in hydrochloric acid and the solution diluted to 100 cu. cm. Ammonia shall be added in slight excess, and the liquid boiled. The lime shall then be reprecipitated by ammonium oxalate, allowed to stand until settled, filtered and washed. The combined filtrates from the calcium precipitates shall be acidified with hydrochloric acid, concentrated on the steam bath to about 150 cu. cm., and made slightly alkaline with ammonium hydroxide, boiled and filtered (to remove a little aluminum and iron and perhaps calcium). When cool, 10 cu. cm. of saturated solution of sodium-ammonium-hydrogen phosphate shall be added with constant stirring. When the crystallin ammonium-magnesium orthophosphate has formed, ammonia shall be added in moderate excess. The solution shall be set aside for several hours in a cool place, filtered and washed with water containing 2.5% of NH_3. The precipitate shall be dissolved in a small quantity of hot hydrochloric acid, the solution diluted to about 100 cu. cm., 1 cu. cm. of a saturated solution of sodium-ammonium-hydrogen phosphate added, and ammonia, drop by drop, with constant stirring, until the precipitate is again formed as described and the ammonia is in moderate excess. The precipitate shall then be allowed to stand about 2 hours, filtered and washed as before. The paper and contents shall be placed in a weighed platinum crucible, the paper slowly charred, and the resulting carbon carefully burned off. The precipitate shall then be ignited to constant weight over a Meker burner, or a blast not strong enough to soften or melt the pyrophosphate. The weight of magnesium pyrophosphate obtained multiplied by 72.5 gives the percentage of magnesia. The precipitate so obtained always contains some calcium and usually small quantities of iron, aluminum, and manganese as phosphates.

27.—*Permissible Variation.*—A permissible variation of 0.4 will be allowed, and all results in excess of the specified limit but within this permissible variation shall be reported as 5.00 per cent.

VIII.—Determination of Specific Gravity

28.—*Apparatus.*—The determination of specific gravity shall be made with a standardized Le Chatelier apparatus which conforms to the requirements illustrated in Fig. 18. This apparatus is standardized by the U. S. Bureau of Standards. Kerosene free from water, or benzine not lighter than 62° Baumé, shall be used in making this determination.

29.—*Method.*—The flask shall be filled with either of these liquids to a point on the stem between zero and 1 cu. cm., and 64 grammes of cement, of the same temperature as the liquid, shall be slowly introduced, taking care that the cement does not adhere to the inside of the flask above the liquid and to free the cement from air by rolling the flask in an inclined position. After all the cement is introduced, the level of the liquid will rise to some division of the graduated neck; the difference between readings is the volume displaced by 64 grammes of the cement.

The specific gravity shall then be obtained from the formula:

$$\text{Specific gravity} = \frac{\text{Weight of cement, in grammes}}{\text{Displaced volume, in cubic centimeters}}$$

30.—The flask, during the operation, shall be kept immersed in water, in order to avoid variations in the temperature of the liquid in the flask, which shall not exceed 0.5° cent. The results of repeated tests should agree within 0.01.

31.—The determination of specific gravity shall be made on the cement as received; if it falls below 3.10, a second determination shall be made after igniting the sample as described in Section 20.

IX.—Determination of Fineness

32.—*Apparatus.*—Wire cloth for standard sieves for cement shall be woven (not twilled) from brass, bronze, or other suitable wire, and mounted without distortion on frames not less than 1½ in. below the top of the frame. The sieve frames shall be circular, approximately 8 in. in diameter, and may be provided with a pan and cover.

33.—A standard No. 200 sieve is one having nominally an 0.0029-in. opening and 200 wires per inch standardized by the U. S. Bureau of Standards, and conforming to the following requirements:

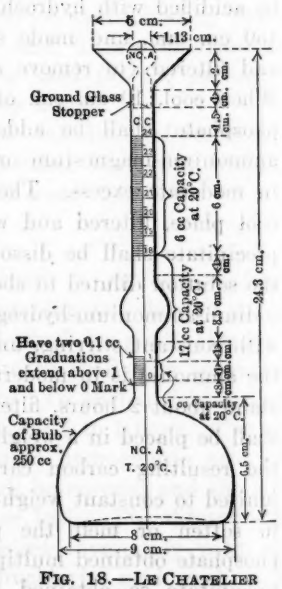


Fig. 18.—Le Chatelier Apparatus.

The No. 200 sieve should have 200 wires per inch, and the number of wires in any whole inch shall not be outside the limits of 192 to 208. No opening between adjacent parallel wires shall be more than 0.0050 in. in width. The diameter of the wire should be 0.0021 in. and the average

diameter shall not be outside the limits 0.0019 to 0.0023 in. The value of the sieve as determined by sieving tests made in conformity with the standard specification for these tests on a standardized cement which gives a residue of 25 to 20% on the No. 200 sieve, or on other similarly graded material, shall not show a variation of more than 1.5% above or below the standards maintained at the Bureau of Standards.

34.—*Method.*—The test shall be made with 50 grammes of cement. The sieve shall be thoroughly clean and dry. The cement shall be placed on the No. 200 sieve, with pan and cover attached, if desired, and shall be held in one hand in a slightly inclined position so that the sample will be well distributed over the sieve, at the same time gently striking the side about 150 times per min. against the palm of the other hand on the up stroke. The sieve shall be turned every 25 strokes about one-sixth of a revolution in the same direction. The operation shall continue until not more than 0.05 gramme passes through in 1 min. of continuous sieving. The fineness shall be determined from the weight of the residue on the sieve expressed as a percentage of the weight of the original sample.

35.—Mechanical sieving devices may be used, but the cement shall not be rejected if it meets the fineness requirement when tested by the hand method described in Section 34.

X.—Mixing Cement Pastes and Mortars

36.—*Method.*—The quantity of dry material to be mixed at one time shall not exceed 1 000 grammes nor be less than 500 grammes. The proportions of cement or cement and sand shall be stated by weight in grammes of the dry materials; the quantity of water shall be expressed in cubic centimeters (1 cu. cm. of water = 1 gramme). The dry materials shall be weighed, placed upon a non-absorbent surface, thoroughly mixed dry if sand is used, and a crater formed in the center, into which the proper percentage of clean water shall be poured; the material on the outer edge shall be turned into the crater by the aid of a trowel. After an interval of ½ min. for the absorption of the water, the operation shall be completed by continuous, vigorous mixing, squeezing, and kneading with the hands for at least 1 min.* During the operation of mixing, the hands should be protected by rubber gloves.

37.—The temperature of the room and the mixing water shall be maintained as nearly as practicable at 21° cent. (70° Fahr.).

XI.—Normal Consistency

38.—*Apparatus.*—The Vicat apparatus consists of a frame, *A* (Fig. 19) bearing a movable rod, *B*, weighing 300 grammes, one end, *C*, being 1 cm. in diameter for a distance of 6 cm., the other having a removable needle, *D*, 1 mm.

* In order to secure uniformity in the results of tests for the time of setting and tensile strength the manner of mixing described should be carefully followed. At least 1 min. is necessary to obtain the desired plasticity which is not appreciably affected by continuing the mixing for several minutes. The exact time necessary is dependent on the personal equation of the operator. The error in mixing should be on the side of over-mixing.

in diameter, 6 cm. long. The rod is reversible, and can be held in any desired position by a screw, *E*, and has midway between the ends a mark, *F*, which moves under a scale (graduated to millimeters) attached to the frame, *A*. The paste is held in a conical, hard-rubber ring, *G*, 7 cm. in diameter at the base, 4 cm. high, resting on a glass plate, *H*, about 10 cm. square.

39.—*Method.*—In making the determination, 500 grammes of cement, with a measured quantity of water, shall be kneaded into a paste, as described in Section 36, and quickly formed into a ball with the hands, completing the operation by tossing it six times from one hand to the other, maintained about

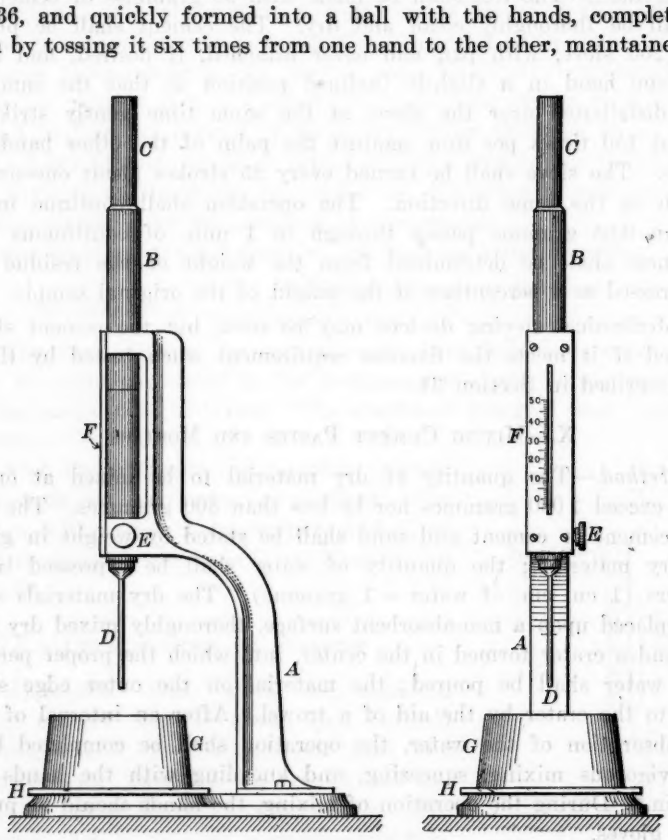


FIG. 19.—VICAT APPARATUS.

6 in. apart; the ball resting in the palm of one hand shall be pressed into the larger end of the rubber ring held in the other hand, completely filling the ring with paste; the excess at the larger end shall then be removed by a single movement of the palm of the hand; the ring shall then be placed on its larger end on a glass plate and the excess paste at the smaller end sliced off at the top of the ring by a single oblique stroke of a trowel held at a slight angle with the top of the ring. During these operations care shall be taken not to compress the paste. The paste confined in the ring, resting on the plate, shall be placed under the rod, the larger end of which shall be brought in contact with the surface of the paste; the scale shall be then read, and the rod quickly

released. The paste shall be of normal consistency when the rod settles to a point 10 mm. below the original surface in ½ min. after being released. The apparatus shall be free from all vibrations during the test. Trial pastes shall be made with varying percentages of water until the normal consistency is obtained. The amount of water required shall be expressed in percentage by weight of the dry cement.

40.—The consistency of standard mortar shall depend on the amount of water required to produce a paste of normal consistency from the same sample of cement. Having determined the normal consistency of the sample, the consistency of standard mortar made from the same sample shall be as indicated in Table 8, the values being in percentage of the combined dry weights of the cement and standard sand.

TABLE 8.—PERCENTAGE OF WATER FOR STANDARD MORTARS

Percentage of water for neat cement paste of normal consistency.	Percentage of water for one cement, three standard Ottawa sand.	Percentage of water for neat cement paste of normal consistency.	Percentage of water for one cement, three standard Ottawa sand.
15	9.0	23	10.8
16	9.2	24	10.5
17	9.3	25	10.7
18	9.5	26	10.8
19	9.	27	11.0
20	9.8	28	11.2
21	10.0	29	11.3
22	10.2	30	11.5

XII.—DETERMINATION OF SOUNDNESS*

41.—*Apparatus.*—A steam apparatus, which can be maintained at a temperature between 98 and 100° cent., or one similar to that shown in Fig. 20, is recommended. The capacity of this apparatus may be increased by using a rack for holding the pats in a vertical or inclined position.

42.—*Method.*—A pat from cement paste of normal consistency about 3 in. in diameter, ½ in. thick at the center, and tapering to a thin edge, shall be made on clean glass plates about 4 in. square, and stored in moist air for 24 hours. In moulding the pat, the cement paste shall first be flattened on the glass and the pat then formed by drawing the trowel from the outer edge toward the center.

43.—The pat shall then be placed in an atmosphere of steam at a temperature between 98 and 100° cent. upon a suitable support 1 in. above boiling water for 5 hours.

44.—Should the pat leave the plate, distortion may be detected best with a straight edge applied to the surface which was in contact with the plate.

XIII.—DETERMINATION OF TIME OF SETTING

45.—The following are alternate methods, either of which may be used as ordered:

* Unsoundness is usually manifested by change in volume which causes distortion, cracking, checking, or disintegration. Pats improperly made or exposed to drying may develop what are known as shrinkage cracks within the first 24 hours and are not an indication of unsoundness. These conditions are illustrated in Fig. 21. The failure of the pats to remain on the glass or the cracking of the glass to which the pats are attached does not necessarily indicate unsoundness.

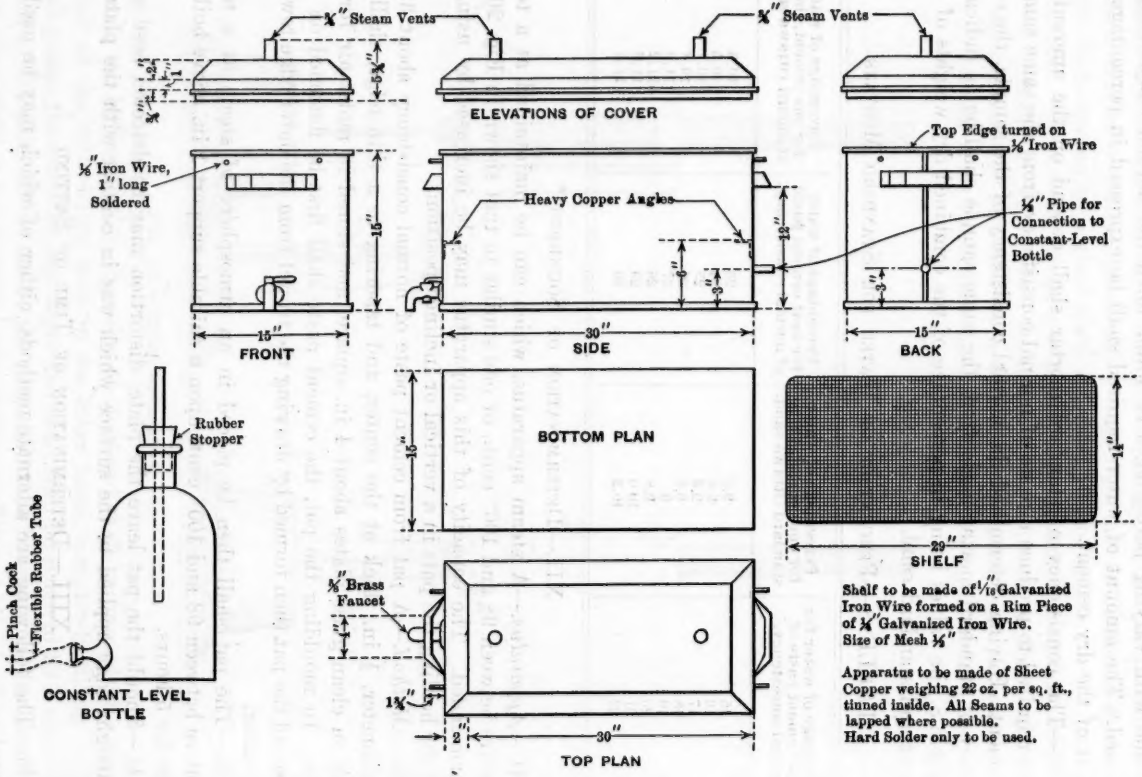

¼" Steam Vents
¼" Steam Vents
ELEVATIONS OF COVER
⅛" Iron Wire, 1" long Soldered
Heavy Copper Angles
Top Edge turned on ⅛" Iron Wire
½" Pipe for Connection to Constant-Level Bottle
15"
12"
6"
3"
FRONT
SIDE
BACK
Rubber Stopper
Pinch Cock
Flexible Rubber Tube
CONSTANT LEVEL BOTTLE
BOTTOM PLAN
SHELF
29"
¼" Brass Faucet
1¼"
2"
30"
TOP PLAN
Shelf to be made of 1/10" Galvanized Iron Wire formed on a Rim Piece of ¼" Galvanized Iron Wire. Size of Mesh ½"
Apparatus to be made of Sheet Copper weighing 22 oz. per sq. ft., tinned inside. All Seams to be lapped where possible. Hard Solder only to be used.

FIG. 20.—APPARATUS FOR MAKING SOUNDNESS TEST OF CEMENT.

TOP PLAN

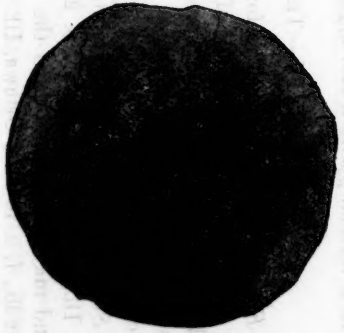
Hard Solder only to be used.

FIG. 20.—APPARATUS FOR MAKING SOUNDNESS TEST OF CEMENT.

Shrinkage

Shrinkage

Distortion



Cracking

Checking

Disintegration

FIG. 21.—TYPICAL FAILURES IN SOUNDNESS TEST.

46.—*Vicat Apparatus.*—The time of setting shall be determined with the Vicat apparatus described in Section 38. (See Fig. 19.)

47.—*Vicat Method.*—A paste of normal consistency shall be moulded in the hard-rubber ring, *G*, as described in Section 39, and placed under the rod, *B*, the smaller end of which shall then be carefully brought in contact with the surface of the paste, and the rod quickly released. The initial set shall be said to have occurred when the needle ceases to pass a point 5 mm. above the glass plate in ½ min. after being released; and the final set, when the needle does not sink visibly into the paste. The test pieces shall be kept in moist air during the test. This may be accomplished by placing them on a rack over water contained in a pan and covered by a damp cloth, kept from contact with them by means of a wire screen; or they may be stored in a moist closet. Care shall be taken to keep the needle clean, as the collection of cement on the sides of the needle retards the penetration, while cement on the point may increase the penetration. The time of setting is affected not only by the percentage and temperature of the water used and the amount of kneading the paste receives, but by the temperature and humidity of the air, and its determination is therefore only proximate.

48.—*Gillmore Needles.*—The time of setting shall be determined by the Gillmore needles. The Gillmore needles should preferably be mounted as shown in Fig. 22 (*b*).

49.—*Gillmore Method.*—The time of setting shall be determined as follows: A pat of neat cement paste about 3 in. in diameter and ½ in. in thickness with a flat top (Fig. 22 (*a*)), mixed to a normal consistency, shall be kept in moist air at a temperature maintained as nearly as practicable at 21° cent. (70° Fahr.). The cement shall be considered to have acquired its initial set when the pat will bear, without appreciable indentation, the Gillmore needle, $\frac{1}{12}$ in. in diameter, loaded to weigh ¼ lb. The final set has been acquired when the pat will bear without appreciable indentation, the Gillmore needle, $\frac{1}{24}$ in. in diameter, loaded to weigh 1 lb. In making the test, the needles shall be held in a vertical position and applied lightly to the surface of the pat.

XIV.—Tension Tests

50.—*Form of Test Piece.*—The form of test piece shown in Fig. 23 shall be used. The moulds shall be made of non-corroding metal and have sufficient material in the sides to prevent spreading during moulding. Gang moulds when used shall be of the type shown in Fig. 24. Moulds shall be wiped with an oily cloth before using.

51.—*Standard Sand.*—The sand to be used shall be natural sand from Ottawa, Ill., screened to pass a No. 20 sieve and retained on a No. 30 sieve. This sand may be obtained from the Ottawa Silica Company, at a cost of 3 cents per lb., f. o. b. cars, Ottawa, Ill.

52.—This sand, having passed the No. 20 sieve, shall be considered standard when not more than 5 grammes pass the No. 30 sieve after 1 min. continuous sieving of a 500-gramme sample.

53.—The sieves shall conform to the following specifications: The No. 20 sieve shall have between 19.5 and 20.5 wires per whole inch of the warp wires and between 19 and 21 wires per whole inch of the shoot wires. The diameter of the wire should be 0.0165 in. and the average diameter shall not be outside the limits of 0.0160 and 0.0170 in.

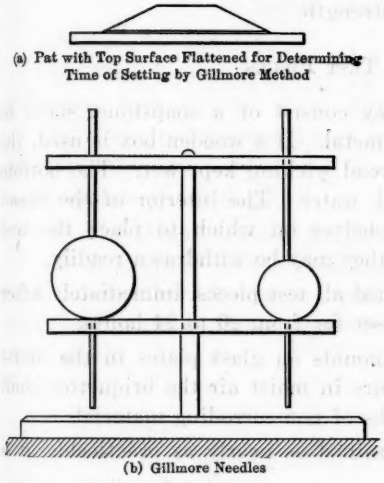


FIG. 22.

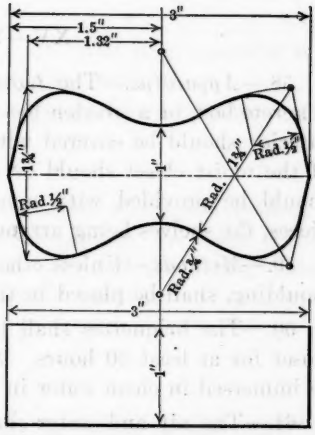


FIG. 23.—DETAILS FOR BRIQUETTE.

The No. 30 sieve shall have between 29.5 and 30.5 wires per whole inch of the warp wires and between 28.5 and 31.5 wires per whole inch of the shoot wires. The diameter of the wire should be 0.0110 in. and the average diameter shall not be outside the limits 0.0105 to 0.0115 in.

54.—*Moulding.*—Immediately after mixing, the standard mortar shall be placed in the moulds, pressed in firmly with the thumbs, and smoothed off with a trowel without ramming. Additional mortar shall be heaped above the mould and smoothed off with a trowel; the trowel shall be drawn over the mould in such a manner as to exert a moderate pressure on the material. The mould shall then be turned over and the operation of heaping, thumbing, and smoothing off repeated.

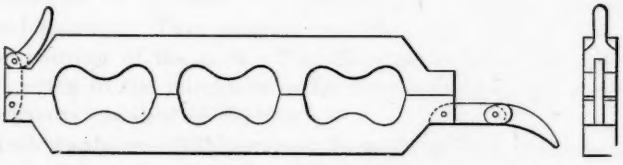


FIG. 24.—GANG MOULD.

55.—*Testing.*—Tests shall be made with any standard machine. The briquettes shall be tested as soon as they are removed from the water. The bearing surfaces of the clips and briquettes shall be free from grains of sand or dirt. The briquettes shall be carefully centered and the load applied continuously at the rate of 600 lb. per min.

56.—Testing machines should be frequently calibrated in order to determine their accuracy.

57.—*Faulty Briquettes.*—Briquettes that are manifestly faulty, or which give strengths differing more than 15% from the average value of all test pieces made from the same sample and broken at the same period, shall not be considered in determining the tensile strength.

XV.—Storage of Test Pieces

58.—*Apparatus.*—The moist closet may consist of a soapstone, slate, or concrete box, or a wooden box lined with metal. If a wooden box is used, the interior should be covered with felt or broad wicking kept wet. The bottom of the moist closet should be covered with water. The interior of the closet should be provided with non-absorbent shelves on which to place the test pieces, the shelves being arranged so that they may be withdrawn readily.

59.—*Methods.*—Unless otherwise specified all test pieces, immediately after moulding, shall be placed in the moist closet for from 20 to 24 hours.

60.—The briquettes shall be kept in moulds on glass plates in the moist closet for at least 20 hours. After 24 hours in moist air the briquettes shall be immersed in clean water in storage tanks of non-corroding material.

61.—The air and water shall be maintained as nearly as practicable at a temperature of 21° cent. (70° Fahr.).

APPENDIX III

STANDARD SPECIFICATIONS FOR BILLET-STEEL CONCRETE REINFORCEMENT BARS*

Serial Designation: A 15-14

1.—*Material Covered.*—

(*a*) These specifications cover three classes of billet-steel concrete reinforcement bars, namely, plain, deformed, and cold-twisted.

(*b*) Plain and deformed bars are of three grades, namely, structural steel, intermediate, and hard.

2.—*Basis of Purchase.*—

(*a*) The structural-steel grade shall be used unless otherwise specified.

(*b*) If desired, cold-twisted bars may be purchased on the basis of tests of the hot-rolled bars before twisting, in which case such tests shall govern and shall conform to the requirements specified for plain bars of structural-steel grade.

I.—Manufacture

3.—*Process.*—

(*a*) The steel shall be made by either or both the following processes, Bessemer or open-hearth.

(*b*) The bars shall be rolled from new billets. No re-rolled material will be accepted.

4.—*Cold-Twisted Bars.*—Cold-twisted bars shall be twisted cold with one complete twist in a length not over 12 times the thickness of the bar.

II.—Chemical Properties and Tests

5.—*Chemical Composition.*—The steel shall conform to the following requirements as to chemical composition:

Phosphorus	Bessemer	not over 0.10 per cent.
	Open-hearth	" " 0.05 "

6.—*Ladle Analyses.*—An analysis of each melt of steel shall be made by the manufacturer to determine the percentages of carbon, manganese, phosphorus, and sulphur. This analysis shall be made from a test ingot taken during the pouring of the melt. The chemical composition thus determined shall be reported to the purchaser or his representative, and shall conform to the requirements specified in Section 5.

7.—*Check Analyses.*—Analyses may be made by the purchaser from finished bars representing each melt of open-hearth steel, and each melt, or lot of 10 tons, of Bessemer steel. The phosphorus content thus determined shall not exceed that specified in Section 5 by more than 25 per cent.

* These specifications are issued by the American Society for Testing Materials under the fixed designation A 15; the final number indicates the year of original adoption as standard, or in the case of revision, the year of last revision. Adopted, 1911; revised, 1912, 1913, 1914.

III.—Physical Properties and Tests

8.—*Tension Tests.*—

(*a*) The bars shall conform to the following requirements (Table 9) as to tensile properties:

TABLE 9.—Tensile Properties

Properties considered.	Plain Bars.			Deformed Bars.			Cold-twisted bars.
	Structural-steel grade.	Intermediate grade.	Hard grade.	Structural-steel grade.	Intermediate grade.	Hard grade.	
Tensile strength, in pounds per square inch....	55 000 to 70 000	70 000 to 85 000	80 000 min.	55 000 to 70 000	70 000 to 85 000	80 000 min.	Recorded only.
Yield point, minimum, in pounds per square inch.	33 000	40 000	50 000	33 000	40 000	50 000	55 000
Elongation in 8 in., minimum, percentage.....	1 400 000* / Tens. str.	1 300 000* / Tens. str.	1 200 000* / Tens. str.	1 250 000* / Tens. str.	1 125 000* / Tens. str.	1 000 000* / Tens. str.	5

* See Section 9.

(*b*) The yield point shall be determined by the drop of the beam of the testing machine.

9.—*Modifications in Elongation.*—

(*a*) For plain and deformed bars over $\frac{3}{4}$ in. in thickness or diameter, a deduction from the percentages of elongation specified in Section 8 (*a*) of 0.25% shall be made for each increase of $\frac{1}{32}$ in. of the specified thickness or diameter above $\frac{3}{4}$ in.

(*b*) For plain and deformed bars under $\frac{7}{16}$ in. in thickness or diameter, a deduction from the percentages of elongation specified in Section 8 (*a*) of 0.5% shall be made for each decrease of $\frac{1}{32}$ in. of the specified thickness or diameter below $\frac{7}{16}$ in.

10.—*Bend Tests.*—The test specimen shall bend cold around a pin without cracking on the outside of the bent portion, as follows (Table 10):

TABLE 10.—Bend-Test Requirements*

Thickness or diameter of bar.	Plain Bars.			Deformed Bars.			Cold-twisted bars.
	Structural-steel grade.	Intermediate grade.	Hard grade.	Structural-steel grade.	Intermediate grade.	Hard grade.	
Under $\frac{3}{4}$ in.....	180° $d = t$	180° $d = 2t$	180° $d = 3t$	180° $d = t$	180° $d = 3t$	180° $d = 4t$	180° $d = 2t$
$\frac{3}{4}$ in. or over.....	180° $d = t$	90° $d = 2t$	90° $d = 3t$	180° $d = 2t$	90° $d = 3t$	90° $d = 4t$	180° $d = 3t$

* d = the diameter of pin about which the specimen is bent; t = the thickness or diameter of the specimen.

11.—*Test Specimens.*—

(*a*) Tension and bend-test specimens for plain and deformed bars shall be taken from the finished bars, and shall be of the full thickness or diameter

of bars as rolled; except that the specimens for deformed bars may be machined for a length of at least 9 in., if deemed necessary by the manufacturer to obtain uniform cross-section.

(*b*) Tension and bend-test specimens for cold-twisted bars shall be taken from the finished bars, without further treatment, except as specified in Section 2 (*b*).

12.—*Number of Tests.*—

(*a*) One tension and one bend test shall be made from each melt of open-hearth steel, and from each melt, or lot of 10 tons, of Bessemer steel; except that if material from one melt differs $\frac{3}{8}$ in. or more in thickness or diameter, one tension and one bend test shall be made from both the thickest and the thinnest material rolled.

(*b*) If any test specimen shows defective machining or develops flaws, it may be discarded and another specimen substituted.

(*c*) If the percentage of elongation of any tension-test specimen is less than that specified in Section 8 (*a*) and any part of the fracture is outside the middle third of the gauge length, as indicated by scribe scratches marked on the specimen before testing, a re-test shall be allowed.

IV.—Permissible Variations in Weight

13.—The weight of any lot of bars shall not vary more than 5% from the theoretical weight of that lot.

V.—Finish

14.—The finished bars shall be free from injurious defects and shall have a workmanlike finish.

VI.—Inspection and Rejection

15.—*Inspection.*—The inspector representing the purchaser shall have free entry, at all times while work on the contract of the purchaser is being performed, to all parts of the manufacturer's works which concern the manufacture of the bars ordered. The manufacturer shall afford the inspector, free of cost, all reasonable facilities to satisfy him that the bars are being furnished in accordance with these specifications. All tests (except check analyses) and inspection shall be made at the place of manufacture prior to shipment, unless otherwise specified, and shall be so conducted as not to interfere unnecessarily with the operation of the works.

16.—*Rejection.*—

(*a*) Unless otherwise specified, any rejection based on tests made in accordance with Section 7 shall be reported within five working days from the receipt of samples.

(*b*) Bars which show injurious defects subsequent to their acceptance at the manufacturer's works will be rejected, and the manufacturer shall be notified.

17.—*Rehearing.*—Samples tested in accordance with Section 7, which represent rejected bars, shall be preserved for two weeks from the date of the test report. In case of dissatisfaction with the results of the tests, the manufacturer may make claim for a rehearing within that time.

APPENDIX IV

STANDARD SPECIFICATIONS FOR RAIL-STEEL CONCRETE REINFORCEMENT BARS*

Serial Designation: A 16-14

1.—*Material Covered.*—These specifications cover three classes of rail-steel concrete reinforcement bars, namely, plain, deformed, and hot-twisted.

I.—Manufacture

2.—*Process.*—The bars shall be rolled from standard section T-rails.

3.—*Hot-Twisted Bars.*—Hot-twisted bars shall have one complete twist in a length not over 12 times the thickness of the bar.

II.—Physical Properties and Tests

4.—*Tension Tests.*—

(*a*) The bars shall conform to the following minimum requirements (Table 11) as to tensile properties.

TABLE 11.

Properties considered.	Plain bars.	Deformed and hot-twisted bars.
Tensile strength, in pounds per square inch	80 000	80 000
Yield point, in pounds per square inch	50 000	50 000
Elongation, in 8 in., percentage*	1 200 000 / Tensile strength	1 000 000 / Tensile strength

* See Section 5.

(*b*) The yield point shall be determined by the drop of the beam of the testing machine.

5.—*Modifications in Elongation.*—

(*a*) For bars over $\frac{3}{4}$ in. in thickness or diameter, a deduction from the percentages of elongation specified in Section 4 (*a*) of 0.25% shall be made for each increase of $\frac{1}{32}$ in. of the specified thickness or diameter above $\frac{3}{4}$ in.

(*b*) For bars under $\frac{7}{16}$ in. in thickness or diameter, a deduction from the percentages of elongation specified in Section 4 (*a*) of 0.5% shall be made for each decrease of $\frac{1}{32}$ in. of the specified thickness or diameter below $\frac{7}{16}$ in.

6.—*Bend Tests.*—The test specimen shall bend cold around a pin without cracking on the outside of the bent portion, as follows (Table 12).

7.—*Test Specimens.*—

(*a*) Tension and bend-test specimens for plain and deformed bars shall be taken from the finished bars, and shall be of the full thickness or diameter of bars as rolled; except that the specimens for deformed bars

* These specifications are issued by the American Society for Testing Materials under the fixed designation A 16; the final number indicates the year of original adoption as standard, or in the case of revision, the year of last revision. Adopted, 1913; revised, 1914.

may be machined for a length of at least 9 in., if deemed necessary by the manufacturer, to obtain uniform cross-section.

TABLE 12.*

Thickness or diameter of bar.	Plain bars.	Deformed and hot-twisted bars.
Under ¾ in.	180° $d = 3t$	180° $d = 4t$
¾ in. or over	90° $d = 3t$	90° $d = 4t$

* d = the diameter of pin about which the specimen is bent; t = the thickness or diameter of the specimen.

(*b*) Tension and bend-test specimens for hot-twisted bars shall be taken from the finished bars, without further treatment.

8.—*Number of Tests.*—

(*a*) One tension and one bend test shall be made from each lot of 10 tons or less of each size of bar rolled from rails varying not more than 10 lb. per yd. in nominal weight.

(*b*) If any test specimen shows defective machining or develops flaws, it may be discarded and another specimen substituted.

(*c*) If the percentage of elongation of any tension test specimen is less than that specified in Section 4 (*a*) and any part of the fracture is outside the middle third of the gauge length, as indicated by scribe scratches marked on the specimen before testing, a re-test shall be allowed.

III.—Permissible Variations in Weight

9.—The weight of any lot of bars shall not vary more than 5% from the theoretical weight of that lot.

IV.—Finish

10.—The finished bars shall be free from injurious defects and shall have a workmanlike finish.

V.—Inspection and Rejection

11.—*Inspection.*—The inspector representing the purchaser shall have free entry, at all times while work on the contract of the purchaser is being performed, to all parts of the manufacturer's works which concern the manufacture of the bars ordered. The manufacturer shall afford the inspector, free of cost, all reasonable facilities to satisfy him that the bars are being furnished in accordance with these specifications. All tests and inspection shall be made at the place of manufacture prior to shipment, unless otherwise specified, and shall be so conducted as not to interfere unnecessarily with the operation of the works.

12.—*Rejection.*—Bars which show injurious defects subsequent to their acceptance at the manufacturer's works will be rejected, and the manufacturer shall be notified.

APPENDIX V

STANDARD SPECIFICATIONS FOR STRUCTURAL STEEL FOR BRIDGES*

Serial Designation: A 7-24

1.—*Steel Castings.*—The Standard Specifications for Steel Castings (Serial Designation: A 27), adopted by the American Society for Testing Materials,† shall govern the purchase of steel castings for bridges. Unless otherwise specified, Class B castings, medium grade, shall be used.

I.—Manufacture

2.—*Process.*—The steel shall be made by the open-hearth process.

II.—Chemical Properties and Tests

3.—*Chemical Composition.*—The steel shall conform to the following requirements as to chemical composition:

		Structural Steel.	Rivet Steel.
Phosphorus	Acid	not over 0.06	not over 0.04 per cent.
	Basic	" " 0.04	" " 0.04 "
Sulphur		" " 0.05	" " 0.045 "

4.—*Ladle Analyses.*—An analysis of each melt of steel shall be made by the manufacturer to determine the percentages of carbon, manganese, phosphorus, and sulphur. This analysis shall be made from a test ingot taken during the pouring of the melt. The chemical composition thus determined shall be reported to the purchaser or his representative, and shall conform to the requirements specified in Section 3.

5.—*Check Analyses.*—Analyses may be made by the purchaser from finished material representing each melt. The phosphorus and sulphur content thus determined shall not exceed that specified in Section 3 by more than 25 per cent.

III.—Physical Properties and Tests

6.—*Tension Tests.*—

(*a*) The material shall conform to the following requirements (Table 13) as to tensile properties.

(*b*) In order to meet the required minimum tensile strength of full-size annealed eye-bars, the purchaser may determine the tensile strength to be obtained in specimen tests; the range shall not exceed 14 000 lb. per sq. in. and the maximum shall not exceed 74 000 lb. per sq. in.. The material shall conform to the requirements as to physical properties other than that of tensile strength, specified in Sections 6, 7, and 8 (*b*).

* These specifications are issued by the American Society for Testing Materials under the fixed designation A 7; the final number indicates the year of original adoption as standard, or in the case of revision, the year of last revision. Adopted, 1901; revised, 1905, 1909, 1913, 1914, 1915, 1916, 1921, 1924.

† 1924 Book of A. S. T. M. Standards.

TABLE 13.

Properties considered.	Structural steel.	Rivet steel.
Tensile strength, in pounds per square inch	55 000–65 000*	46 000–56 000
Yield point, minimum, per square inch	0.5 tens. str.	0.5 tens. str.
but in no case less than	30 000 lb. per sq. in.	25 000 lb. per sq. in.
Elongation in 8 in., minimum, percentage	$\frac{1\,500\,000}{\text{Tens. str.}}$†	$\frac{1\,500\,000}{\text{Tens. str.}}$
Elongation in 2 in., minimum, percentage	22	.....

* See Paragraph (*b*).
† See Section 7.

(*c*) The yield point shall be determined by the drop of the beam of the testing machine.

7.—*Modifications in Elongation.*—

(*a*) For structural steel over $\frac{3}{4}$ in. in thickness or diameter, a deduction from the percentage of elongation in 8 in. specified in Section 6 (*a*) of 0.25% shall be made for each increase of $\frac{1}{8}$ in. of the specified thickness or diameter above $\frac{3}{4}$ in., to a minimum of 18 per cent.

(*b*) For structural steel under $\frac{5}{16}$ in. in thickness or diameter, a deduction from the percentage of elongation in 8 in. specified in Section 6 (*a*) of 1.25% shall be made for each decrease of $\frac{1}{8}$ in. in thickness or diameter below $\frac{5}{16}$ in.

8.—*Bend Tests.*—

(*a*) Bend test specimens, except as specified in Paragraphs (*b*) and (*c*), shall stand being bent cold through 180° without cracking on the outside of the bent portion, as follows: For material $\frac{3}{4}$ in. or under in thickness, flat on itself; for material over $\frac{3}{4}$ in. to and including $1\frac{1}{4}$ in. in thickness, around a pin the diameter of which is equal to the thickness of the specimen; and for material over $1\frac{1}{4}$ in. in thickness, around a pin the diameter of which is equal to twice the thickness of the specimen.

(*b*) Bend test specimens for eye-bar flats shall stand being bent cold through 180° without cracking on the outside of the bent portion as follows: For material $\frac{3}{4}$ in. or under in thickness, around a pin the diameter of which is equal to the thickness of the specimen; for material over $\frac{3}{4}$ in. to and including $1\frac{1}{4}$ in. in thickness, around a pin the diameter of which is equal to twice the thickness of the specimen; and for material over $1\frac{1}{4}$ in. in thickness, around a pin the diameter of which is equal to three times the thickness of the specimen.

(*c*) Bend test specimens for rivet steel shall stand being bent cold through 180° flat on themselves without cracking on the outside of the bent portion.

9.—*Test Specimens.*—

(*a*) Test specimens shall be prepared for testing from the material in its rolled or forged condition, except as specified in Paragraphs (*b*) and (*c*).

(*b*) Test specimens for annealed material shall be prepared from the material as annealed for use, or from a short length of a full section similarly treated.

(*c*) Test specimens for rivet bars which have been cold-drawn shall be normalized before testing.

(*d*) Test specimens shall be taken longitudinally and, except as specified in Paragraphs (*f*), (*g*), and (*h*), shall be of the full thickness or section of material as rolled.

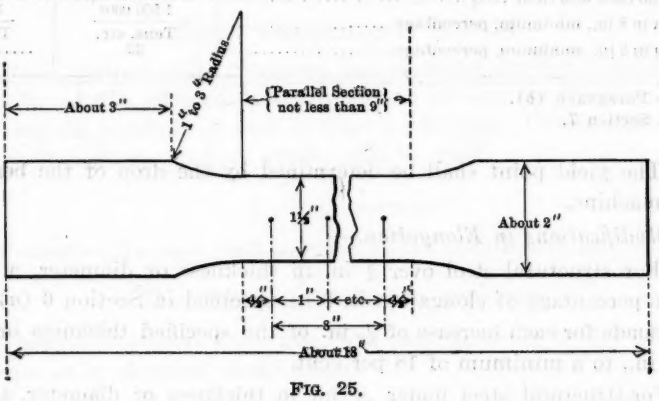


FIG. 25.

(*e*) Test specimens for plates, shapes, and flats may be machined to the form and dimensions shown in Fig. 25, or with both edges parallel. Bend-test specimens for eye-bar flats may have three rolled sides.

(*f*) Tension-test specimens for material over 1½ in. in thickness or diameter, except pins and rollers, may be machined to a thickness or diameter of at least ¾ in. for a length of at least 9 in., or they may conform to the dimensions shown in Fig. 26.

(*g*) Bend test specimens for material over 1½ in. in thickness or diameter, except eye-bar flats, pins, and rollers, may be machined to a thickness or diameter of at least ¾ in. or to 1 by ½ in. in section.

(*h*) Tension-test specimens for pins and rollers shall conform to the dimensions shown in Fig. 26 and bend-test specimens shall be 1 by ½ in. in section.

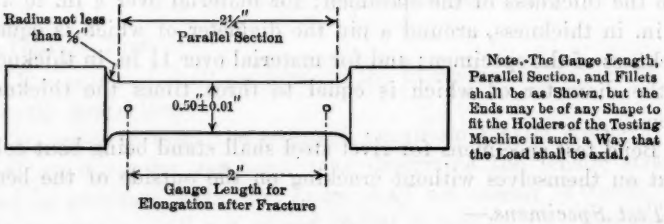

Radius not less than ⅜"
2¼"
Parallel Section
0.50"±0.01"
2"
Gauge Length for Elongation after Fracture
Note.-The Gauge Length, Parallel Section, and Fillets shall be as Shown, but the Ends may be of any Shape to fit the Holders of the Testing Machine in such a Way that the Load shall be axial.

FIG. 26.

(*i*) Test specimens for pins and rollers shall be taken so that the axis is 1 in. from the surface.

(*j*) The machined sides of rectangular bend-test specimens may have the corners rounded to a radius of not over ⅛ in.

10.—*Number of Tests.*—

(*a*) One tension and one bend test shall be made from each melt; except that if material from one melt differs ⅜ in. or more in thickness, one tension and one bend test shall be made from both the thickest and the thinnest material rolled.

(*b*) If any test specimen shows defective machining or develops flaws, it may be discarded and another specimen substituted.

(*c*) If the percentage of elongation of any tension test specimen is less than that specified in Section 6(*a*) and any part of the fracture is more than ¾ in. from the center of the gauge length of a 2-in. specimen or is outside the middle third of the gauge length of an 8-in. specimen, as indicated by scribe scratches marked on the specimen before testing, a re-test shall be allowed.

TABLE 14.—PERMISSIBLE VARIATIONS OF RECTANGULAR PLATES ORDERED TO WEIGHT.*

Ordered weight, in pounds per square foot.	Permissible Variations in Average Weights per Square Foot of Plates for Widths Given, Expressed in Percentages of Ordered Weights.																				Ordered weight, in pounds per square foot.
	Under 48 in.		48 to 60 in., excl.		60 to 72 in., excl.		72 to 84 in., excl.		84 to 96 in., excl.		96 to 108 in., excl.		108 to 120 in., excl.		120 to 132 in., excl.		132 in. or over.			
	Over.	Under.	Over.	Under.	Over.	Under.	Over.	Under.	Over.	Under.	Over.	Under.	Over.	Under.	Over.	Under.	Over.	Under.		
Under 5	5	3	5.5	3	6	3	7	3	...	...	...	...	...	...	...	...	...	...			Under 5
5 to 7.5 excl.	4.5	3	5	3	5.5	3	6	3	...	...	...	...	...	...	...	...	...	...			5 to 7.5 excl.
7.5 " 10 "	4	3	4.5	3	5	3	5.5	3	6	3	7	3	8	3	...	...	...	...			7.5 " 10 "
10 " 12.5 "	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3	7	3	8	3	9	3			10 " 12.5 "
12.5 " 15 "	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3	7	3	8	3			12.5 " 15 "
15 " 17.5 "	2.5	2.5	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3	7	3			15 " 17.5 "
17.5 " 20 "	2.5	2	2.5	2.5	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3			17.5 " 20 "
20 " 25 "	2	2	2.5	2	2.5	2.5	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3			20 " 25 "
25 " 30 "	2	2	2	2	2.5	2	2.5	2.5	3	2.5	3.5	2.5	4	3	4.5	3	5	3			25 " 30 "
30 " 40 "	2	2	2	2	2	2	2.5	2	2.5	2.5	3	2.5	3.5	3	4	3	4.5	3			30 " 40 "
40 or over	2	2	2	2	2	2	2	2	2.5	2	2.5	2.5	3	2.5	3.5	3	4	3			40 or over

* The weight per square foot of individual plates shall not vary from the ordered weight by more than 1⅓ times the amount given in this table.

IV.—PERMISSIBLE VARIATIONS IN WEIGHT AND THICKNESS

11.—The cross-section or weight of each piece of steel shall not vary more than 2.5% from that specified; except in the case of sheared plates, which shall be covered by the following permissible variations. One cubic inch of rolled steel is assumed to weigh 0.2833 lb.

(*a*) *When Ordered to Weight per Square Foot.*—The weight of each lot* in each shipment shall not vary from the weight ordered more than the amount given in Table 14.

* The term, "lot", applied to Table 14 means all the plates of each group width and group weight; as applied to Table 15 it means all the plates of each group width and group thickness.

(*b*) *When Ordered to Thickness.*—The thickness of each plate shall not vary more than 0.01 in. under that ordered.

The overweight of each lot* in each shipment shall not exceed the amount given in Table 15.

TABLE 15.—PERMISSIBLE OVERWEIGHTS OF RECTANGULAR PLATES ORDERED TO THICKNESS.

Ordered thickness, in inches.	PERMISSIBLE EXCESS IN AVERAGE WEIGHTS PER SQUARE FOOT OF PLATES FOR WIDTHS GIVEN, EXPRESSED IN PERCENTAGES OF NOMINAL WEIGHTS.									Ordered thickness, in inches
	Under 48 in.	48 to 60 in., excl.	60 to 72 in., excl.	72 to 84 in., excl.	84 to 96 in., excl.	96 to 108 in., excl.	108 to 120 in., excl.	120 to 132 in., excl.	132 in. or over.	
Under 1/8	9	10	12	14	..	..	..	..	..	Under 1/8
1/8 to 3/16 excl.	8	9	10	12	..	..	..	..	..	1/8 to 3/16 excl.
3/16 " 1/4 "	7	8	9	10	12	..	..	..	..	3/16 " 1/4 "
1/4 " 5/16 "	6	7	8	9	10	12	14	16	19	1/4 " 5/16 "
5/16 " 3/8 "	5	6	7	8	9	10	12	14	17	5/16 " 3/8 "
3/8 " 7/16 "	4.5	5	6	7	8	9	10	12	15	3/8 " 7/16 "
7/16 " 1/2 "	4	4.5	5	6	7	8	9	10	13	7/16 " 1/2 "
1/2 " 5/8 "	3.5	4	4.5	5	6	7	8	9	11	1/2 " 5/8 "
5/8 " 3/4 "	3	3.5	4	4.5	5	6	7	8	9	5/8 " 3/4 "
3/4 " 1 "	2.5	3	3.5	4	4.5	5	6	7	8	3/4 " 1 "
1 or over,	2.5	2.5	3	3.5	4	4.5	5	6	7	1 or over

V.—FINISH

12.—The finished material shall be free from injurious defects and shall have a workmanlike finish.

VI.—MARKING

13.—The name or brand of the manufacturer and the melt number shall be legibly stamped or rolled on all finished material, except that rivet and lattice-bars and other small sections shall, when loaded for shipment, be properly separated and marked for identification. The identification marks shall be legibly stamped on the end of each pin and roller. The melt number shall be legibly marked, by stamping if practicable, on each test specimen.

VII.—INSPECTION AND REJECTION

14.—*Inspection.*—The inspector representing the purchaser shall have free entry, at all times while work on the contract of the purchaser is being performed, to all parts of the manufacturer's works which concern the manufacture of the material ordered. The manufacturer shall afford the inspector, free of cost, all reasonable facilities to satisfy him that the material is being furnished in accordance with these specifications. All tests (except check analyses) and inspection shall be made at the place of manufacture prior to shipment, unless otherwise specified, and shall be so conducted as not to interfere unnecessarily with the operation of the works.

* The term, "lot", applied to Table 14 means all the plates of each group width and group weight; as applied to Table 15 it means all the plates of each group width and group thickness.

15.—*Rejection.*—

(*a*) Unless otherwise specified, any rejection based on tests made in accordance with Section 5 shall be reported within five working days from the receipt of samples.

(*b*) Material which shows injurious defects subsequent to its acceptance at the manufacturer's works will be rejected, and the manufacturer shall be notified.

16.—*Rehearing.*—Samples tested in accordance with Section 5, which represent rejected material, shall be preserved for two weeks from the date of the test report. In case of dissatisfaction with the results of the tests, the manufacturer may make claim for a rehearing within that time.

APPENDIX VI

STANDARD SPECIFICATIONS FOR STRUCTURAL STEEL FOR BUILDINGS*

Serial Designation: A 9-24

I.—Manufacture

1.—*Process.*—

(*a*) Structural steel, except as noted in Paragraph (*b*), shall be made by either or both the following processes: Bessemer or open-hearth.

(*b*) Rivet steel, and steel for plates or angles over $\frac{3}{4}$ in. in thickness, which are to be punched, shall be made by the open-hearth process.

II.—Chemical Properties and Tests

2.—*Chemical Composition.*—The steel shall conform to the following requirements as to chemical composition:

	Structural Steel	Rivet Steel
Phosphorus	Bessemer.. not over 0.10 per cent.	.....
	Open-hearth " " 0.06 "	not over 0.06 per cent.
Sulphur.....	....	" " 0.045 "

3.—*Ladle Analyses.*—

(*a*) A carbon determination shall be made of each melt of Bessemer steel, and determinations for manganese, phosphorus, and sulphur representing the average of the melts applied for each 12-hour period.

(*b*) An analysis of each melt of open-hearth steel shall be made for carbon, manganese, phosphorus, and sulphur.

(*c*) These analyses shall be made by the manufacturer from test ingots taken during the pouring of each melt. The chemical composition thus determined shall be reported to the purchaser or his representative, and shall conform to the requirements specified in Section 2.

4.—*Check Analyses.*—Analyses may be made by the purchaser from finished material representing each melt. The phosphorus and sulphur content thus determined shall not exceed that specified in Section 2 by more than 25 per cent.

III.—Physical Properties and Tests

5.—*Tension Tests.*—

(*a*) The material shall conform to the following requirements (Table 16) as to tensile properties.

(*b*) The yield point shall be determined by the drop of the beam of the testing machine.

* These specifications are issued by the American Society for Testing Materials under the fixed designation A 9; the final number indicates the year of original adoption as standard, or in the case of revision, the year of last revision. Adopted, 1901; revised, 1909, 1913, 1914, 1916, 1921, 1924.

TABLE 16

Properties considered.	Structural steel.	Rivet steel.
Tensile strength, in pounds per square inch...	55 000–65 000	46 000–56 000
Yield point, minimum, per square inch.....	0.5 tens. str.	0.5 tens. str.
but in no case less than.....	30 000 lb. per sq. in.	25 000 lb. per sq. in.
	1 400 000*	1 400 000
Elongation in 8 in., minimum, percentage.....	Tens. str.	Tens. str.
Elongation in 2 in., minimum, percentage.....	22	.....

* See Section 6.

6.—*Modifications in Elongation.*—

(*a*) For structural steel over $\frac{3}{4}$ in. in thickness, a deduction from the percentage of elongation in 8 in. specified in Section 5 (*a*) of 0.25% shall be made for each increase of $\frac{1}{32}$ in. of the specified thickness above $\frac{3}{4}$ in., to a minimum of 18 per cent.

(*b*) For structural steel under $\frac{5}{16}$ in. in thickness, a deduction from the percentage of elongation in 8 in. specified in Section 5 (*a*) of 1.25% shall be made for each decrease of $\frac{1}{32}$ in. of the specified thickness below $\frac{5}{16}$ in.

7.—*Bend Tests.*—

(*a*) Bend test specimens except as specified in Paragraph (*b*), shall stand being bent cold through 180° without cracking on the outside of the bent portion, as follows: For material $\frac{3}{4}$ in. or under in thickness, flat on itself; for material over $\frac{3}{4}$ in. to and including $1\frac{1}{4}$ in. in thickness, around a pin the diameter of which is equal to the thickness of the specimen; and for material over $1\frac{1}{4}$ in. in thickness, around a pin the diameter of which is equal to twice the thickness of the specimen.

(*b*) Bend test specimens for rivet steel shall stand being bent cold through 180° flat on themselves without cracking on the outside of the bent portion.

8.—*Test Specimens.*—

(*a*) Test specimens shall be prepared for testing from the material in its rolled or forged condition, except as specified in Paragraphs (*b*) and (*c*).

(*b*) Test specimens for annealed material shall be prepared from the material as annealed for use, or from a short length of a full section similarly treated.

(*c*) Test specimens for rivet bars which have been cold-drawn shall be normalized before testing.

(*d*) Test specimens shall be taken longitudinally and, except as specified in Paragraphs (*f*), (*g*), and (*h*), shall be of the full thickness or section of material as rolled.

(*e*) Test specimens for plates, shapes, and flats, may be machined to the form and dimensions shown in Fig. 25 (Appendix V), or with both edges parallel.

(*f*) Tension test specimens for material over $1\frac{1}{2}$ in. in thickness or diameter, except pins and rollers, may be machined to a thickness or diameter of at least $\frac{3}{4}$ in. for a length of at least 9 in., or they may conform to the dimensions shown in Fig. 26 (Appendix V).

(*g*) Bend test specimens for material over 1½ in. in thickness or diameter, except pins and rollers, may be machined to a thickness or diameter of at least ¾ in. or to 1 by ½ in. in section.

(*h*) Tension-test specimens for pins and rollers shall conform to the dimensions shown in Fig. 26, and bend-test specimens shall be 1 by ½ in. in section.

(*i*) Test specimens for pins and rollers shall be taken so that the axis is 1 in. from the surface.

(*j*) The machined sides of rectangular bend-test specimens may have the corners rounded to a radius not over ⅛ in.

9.—*Number of Tests.*—

(*a*) One tension and one bend test shall be made from each melt; except that if material from one melt differs ¾ in. or more in thickness, one tension and one bend test shall be made from both the thickest and the thinnest material rolled.

(*b*) If any test specimen shows defective machining or develops flaws, it may be discarded and another specimen substituted.

(*c*) If the percentage of elongation of any tension-test specimen is less than that specified in Section 5 (*a*) and any part of the fracture is more than ¾ in. from the center of the gauge length of a 2-in. specimen or is outside the middle third of the gauge length of an 8-in. specimen, as indicated by scribe scratches marked on the specimen before testing, a re-test shall be allowed.

IV. Permissible Variations in Weight and Thickness

10.—*Permissible Variations.*—The cross-section or weight of each piece of steel shall not vary more than 2.5% from that specified; except in the case of sheared plates, which shall be covered by the following permissible variations. One cubic inch of rolled steel is assumed to weigh 0.2833 lb.

(*a*) *When Ordered to Weight per Square Foot.*—The weight of each lot* in each shipment shall not vary from the weight ordered more than the amount given in Table 17.

(*b*) *When Ordered to Thickness.*—The thickness of each plate shall not vary more than 0.01 in. under that ordered.

The overweight of each lot* in each shipment shall not exceed the amount given in Table 18.

V.—Finish

11.—The finished material shall be free from injurious defects and shall have a workmanlike finish.

VI.—Marking

12.—The name or brand of the manufacturer and the melt number shall be legibly stamped or rolled on all finished material, except that rivet and

* The term, "lot", as applied to Table 17 means all the plates of each group width and group weight; as applied to Table 18, it means all the plates of each group width and group thickness.

lattice-bars and other small sections shall, when loaded for shipment, be properly separated and marked for identification. The identification marks shall be legibly stamped on the end of each pin and roller. The melt number shall be legibly marked, by stamping if practicable, on each test specimen.

TABLE 17.—PERMISSIBLE VARIATIONS OF RECTANGULAR PLATES ORDERED TO WEIGHT.*

Ordered weight, in pounds per square foot.	Permissible Variations in Average Weights per Square Foot of Plates for Widths Given, Expressed in Percentages of Ordered Weights.																		Ordered weight, in pounds per square foot.
	Under 48 in.		48 to 60 in., excl.		60 to 72 in., excl.		72 to 84 in., excl.		84 to 96 in., excl.		96 to 108 in., excl.		108 to 120 in., excl.		120 to 132 in., excl.		132 in. or over.		
	Over.	Under.	Over.	Under.	Over.	Under.	Over.	Under.	Over.	Under.	Over.	Under.	Over.	Under.	Over.	Under.	Over.	Under.	
Under 5	5	3	5.5	3	6	3	7	3	...	...	...	...	...	...	...	...	...	...	Under 5
5 to 7.5 excl.	4.5	3	5	3	5.5	3	6	3	...	...	...	...	...	...	...	...	...	...	5 to 7.5 excl.
7.5 " 10 "	4	3	4.5	3	5	3	5.5	3	6	3	7	3	8	3	...	...	...	...	7.5 " 10 "
10 " 12.5 "	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3	7	3	8	3	9	3	10 " 12.5 "
12.5 " 15 "	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3	7	3	8	3	12.5 " 15 "
15 " 17.5 "	2.5	2.5	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3	7	3	15 " 17.5 "
17.5 " 20 "	2.5	2	2.5	2.5	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3	17.5 " 20 "
20 " 25 "	2	2	2.5	2	2.5	2.5	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3	20 " 25 "
25 " 30 "	2	2	2	2	2.5	2	2.5	2.5	3	2.5	3.5	3	4	3	4.5	3	5	3	25 " 30 "
30 " 40 "	2	2	2	2	2	2	2.5	2	2.5	2.5	3	2.5	3.5	3	4	3	4.5	3	30 " 40 "
40 or over	2	2	2	2	2	2	2	2	2.5	2	2.5	2.5	3	2.5	3.5	3	4	3	40 or over.

* The weight per square foot of individual plates shall not vary from the ordered weight by more than 1⅓ times the amount given in this table.

TABLE 18.—PERMISSIBLE OVERWEIGHTS OF RECTANGULAR PLATES ORDERED TO THICKNESS.

Ordered thickness, in inches.	Permissible Excess in Average Weights per Square Foot of Plates for Widths Given, Expressed in Percentages of Nominal Weights.									Ordered thickness, in inches.
	Under 48 in.	48 to 60 in., excl.	60 to 72 in., excl.	72 to 84 in., excl.	84 to 96 in., excl.	96 to 108 in., excl.	108 to 120 in., excl.	120 to 132 in., excl.	132 in. or over.	
Under ⅛	9	10	12	14	..	..	..	..	..	Under ⅛
⅛ to 3/16 excl.	8	9	10	12	..	..	..	..	..	⅛ to 3/16 excl.
3/16 " ¼ "	7	8	9	10	12	..	..	..	..	3/16 " ¼ "
¼ " 5/16 "	6	7	8	9	10	12	14	16	19	¼ " 5/16 "
5/16 " ⅜ "	5	6	7	8	9	10	12	14	17	5/16 " ⅜ "
⅜ " 7/16 "	4.5	5	6	7	8	9	10	12	15	⅜ " 7/16 "
7/16 " ½ "	4	4.5	5	6	7	8	9	10	13	7/16 " ½ "
½ " 9/16 "	3.5	4	4.5	5	6	7	8	9	11	½ " 9/16 "
9/16 " ⅝ "	3	3.5	4	4.5	5	6	7	8	9	9/16 " ⅝ "
⅝ " ¾ "	2.5	3	3.5	4	4.5	5	6	7	8	⅝ " ¾ "
¾ " 1 "	2.5	2.5	3	3.5	4	4.5	5	6	7	¾ " 1 "
1 or over	2.5	2.5	3	3.5	4	4.5	5	6	7	1 or over

VII.—INSPECTION AND REJECTION

13.—*Inspection.*—The inspector representing the purchaser shall have free entry, at all times while work on the contract of the purchaser is being per-

formed, to all parts of the manufacturer's works which concern the manufacture of the material ordered. The manufacturer shall afford the inspector, free of cost, all reasonable facilities to satisfy him that the material is being furnished in accordance with these specifications. All tests (except check analyses) and inspection shall be made at the place of manufacture prior to shipment, unless otherwise specified, and shall be so conducted as not to interfere unnecessarily with the operation of the works.

14.—*Rejection.*—

(*a*) Unless otherwise specified, any rejection based on tests made in accordance with Section 4 shall be reported within five working days from the receipt of samples.

(*b*) Material which shows injurious defects subsequent to its acceptance at the manufacturer's works will be rejected, and the manufacturer shall be notified.

15.—*Rehearing.*—Samples tested in accordance with Section 4, which represent rejected material, shall be preserved for two weeks from the date of the test report. In case of dissatisfaction with the results of the tests, the manufacturer may make claim for a rehearing within that time.

APPENDIX VII

TENTATIVE SPECIFICATIONS FOR COLD-DRAWN STEEL WIRE FOR CONCRETE REINFORCEMENT*

Serial Designation: A. 82-21 T

1.—*Material Covered.*—These specifications cover cold-drawn steel wire to be used as such, or in fabricated form, for the reinforcement of concrete, in gauges not less than 0.080 in. nor greater than 0.500 in.

2.—*Basis of Purchase.*—When wire is ordered by gauge number the following relation between number and diameter, in inches, shall apply unless otherwise specified:

Gauge number.	Equivalent Diameter, in inches.	Gauge number.	Equivalent Diameter, in inches.
0000000	0.4900	5	0.2070
000000	0.4615	6	0.1920
00000	0.4305	7	0.1770
0000	0.3938	8	0.1620
000	0.3625	9	0.1483
00	0.3310	10	0.1350
0	0.3065	11	0.1205
1	0.2830	12	0.1055
2	0.2625	13	0.0915
3	0.2437	14	0.0800
4	0.2253		

I.—Manufacture

3.—*Process.*—

(*a*) The steel shall be made by either or both the following processes: Bessemer or open hearth.

(*b*) *Drawing.*—The wire shall be cold-drawn from rods hot-rolled from billets.

II.—Physical Properties and Tests

4.—*Tension Tests.*—

(*a*) The wire, except as specified in Paragraphs (*b*) and (*c*) shall conform to the following minimum requirements as to tensile properties:

Tensile strength, in pounds per square inch	80 000
Reduction of area, percentage	30

NOTE.—Criticisms of these Tentative Specifications are solicited and should be directed to Mr. G. H. Woodroffe, Secretary of Committee A-1 on Steel, Parkesburg Iron Co., Parkesburg, Pa.

* This is a Tentative Standard only, published by the American Society for Testing Materials for the purpose of eliciting criticism and suggestions. It is not a Standard of the Society and until its adoption as Standard, it is subject to revision. Issued, 1921.

(*b*) For wire to be used in the fabrication of mesh a minimum tensile strength of 70 000 lb. per sq. in. shall be permitted.

(*c*) For wire testing over 100 000 lb. per sq. in. tensile strength, the reduction of area shall be not less than 25 per cent.

5.—*Bend Tests.*—The test specimen shall withstand being bent cold through 180° without cracking on the outside of the bent portion, as follows:

For wire, 0.3 in. in diameter or under, around a pin the diameter of which is equal to the diameter of the specimen.

For wire over 0.3 in. in diameter, around a pin the diameter of which is equal to twice the diameter of the specimen.

6.—*Test Specimens.*—Tension and bend-test specimens shall be of the full-size section of the wire as drawn.

7.—*Number of Tests.*—

(*a*) One tension and one bend test shall be made for each 10 tons or less of each size of wire.

(*b*) If any test specimen shows defects or develops flaws, it may be discarded and another specimen substituted.

III.—Permissible Variations in Gauge

8.—The diameter of the wire shall not vary more than 0.003 in. from the size ordered.

IV.—Workmanship and Finish

9.—The finished wire shall be free from injurious defects and shall have a workmanlike finish with smooth surface.

V.—Inspection and Rejection

10.—*Inspection.*—The manufacturer shall afford the inspector, without charge, all reasonable facilities to satisfy him that the wire is being furnished in accordance with these specifications. All tests and inspection shall be made at the place of manufacture prior to shipment, unless otherwise specified, and shall be so conducted as not to interfere unnecessarily with the operation of the works.

11.—*Rejection.*—Wire which shows injurious defects subsequent to its acceptance at the manufacturer's works will be rejected, and the manufacturer shall be notified.

APPENDIX VIII

STANDARD METHOD OF TEST FOR SIEVE ANALYSIS OF AGGREGATES FOR CONCRETE*

Serial Designation: C 41-24

1.—*Sampling.*—A representative test sample of the aggregate shall be selected by quartering or by use of a sampler, which after drying will give not less than the following:

(*a*) Fine aggregate, 500 grammes.

(*b*) Coarse aggregate, or a mixture of fine and coarse aggregates, weight, in grammes, 3 000 times size of largest sieve required, measured in inches.

TABLE 19.

Sieve number* or size, in inches.	Sieve opening.		Wire diameter.		Tolerance, percentage.			
	Inches.	Millimeters.	Inches.	Millimeters.	Average opening.	Wire diameter. Under.	Wire diameter. Over.	Maximum opening.
No. 100	0.0059	0.149	0.0040	0.102	6	15	35	40
No. 50	0.0117	0.297	0.0074	0.188	6	15	35	40
No. 30	0.0232	0.59	0.0130	0.33	5	15	30	25
No. 16	0.0469	1.19	0.0213	0.54	3	15	30	10
No. 8	0.0937	2.38	0.0331	0.84	3	15	30	10
No. 4	0.187	4.76	0.050	1.27	3	15	30	10
¾-in	0.375	9.5	0.092	2.33	3	10	10	10
¾-in	0.75	19.0	0.135	3.42	3	10	10	10
1-in	1.00	25.4	0.162	4.12	3	10	10	10
1½-in	1.50	38.0	0.177	4.50	3	10	10	10
2-in	2.00	50.8	0.192	4.88	3	10	10	10
3-in	3.00	76.0	0.25	6.3	3	10	10	10

* The requirements for Sieves No. 100 to No. 4 conform to the requirements of the U. S. Standard Sieve series as given in the U. S. Bureau of Standards *Letter Circular No. 74.* The liberal tolerances will permit the use of certain sieves which do not exactly correspond to the numbers given in the table.

2.—*Treatment of Sample.*—The sample shall be dried at not over 110° cent. (230° Fahr.) to constant weight.

3.—*Sieves.*—

(*a*) The sieves shall be of square-mesh wire cloth and shall be mounted on substantial frames constructed in a manner that will prevent loss of material during sifting.

(*b*) The size of wire and sieve openings shall be as given in Table 19.

4.—*Procedure.*—

(*a*) The sample shall be separated into a series of sizes by means of the sieves specified in Section 3. Sifting shall be continued until not more than 1% by weight of the sample passes any sieve during 1 min.

* This method is issued by the American Society for Testing Materials under the fixed designation C 41; the final number indicates the year of original adoption as standard, or in the case of revision, the year of last revision. Proposed as Tentative, 1921; adopted, 1922; revised, 1924.

(*b*) Each size shall be weighed on a balance or scale which is sensitive to $\frac{1}{1\,000}$ of the weight of the test sample.

(*c*) The percentage by weight of the total sample which is finer than each of the sieves shall be computed.

5.—*Report.*—

(*a*) The percentages in sieve analysis shall be reported to the nearest whole number.

(*b*) If more than 15% of a fine aggregate is coarser than the No. 4 sieve, or more than 15% of a coarse aggregate is finer than the No. 4 sieve, the sieve analysis of the portions finer and coarser than this sieve shall be reported separately.

APPENDIX IX

TENTATIVE METHOD OF DECANTATION TEST FOR SAND AND OTHER FINE AGGREGATES*

Serial Designation: D 136-22 T.

1.—*Scope.*—This method of test covers the determination of the total quantity of silt, loam, clay, etc., in sand and other fine aggregates.†

2.—*Apparatus.*—The pan or vessel to be used in the determination shall be approximately 9 in. (230 mm.) in diameter and not less than 4 in. (102 mm.) in depth.

3.—*Treatment of Sample.*—The sample must contain sufficient moisture to prevent segregation and shall be thoroughly mixed. A representative portion of the sample sufficient to yield approximately 500 grammes of dried material, shall then be dried to a constant weight at a temperature not exceeding 110° cent. (230° Fahr.).

4.—*Procedure.*—The dried material shall be placed in the pan and sufficient water added to cover the sample (about 225 cu. cm.). The contents of the pan shall be agitated vigorously for 15 sec., and then be allowed to settle for 15 sec., after which the water shall be poured off, care being taken not to pour off any sand. This operation shall be repeated until the wash water is clear. As a precaution, the wash water shall be poured through a 200-mesh sieve and any material retained thereon returned to the washed sample. The washed sand shall be dried to a constant weight at a temperature not exceeding 100° cent. (230° Fahr.) and weighed.

5.—*Calculation of Results.*—The results shall be calculated from the formula:

$$\text{Percentage of silt, clay, loam, etc.} = \frac{\text{Original dry weight} - \text{weight after washing}}{\text{Original dry weight}} \times 100$$

6.—*Check Determination.*—When check determinations are desired, the wash water shall be evaporated to dryness, the residue weighed, and the percentage calculated from the formula:

$$\text{Percentage of silt, loam, clay, etc.} = \frac{\text{Weight of residue}}{\text{Original dry weight}} \times 100$$

NOTE.—Criticisms of this Tentative Method are solicited and should be directed to Mr. J. C. Pearson, Secretary of Committee C-9 on Concrete and Concrete Aggregates, Bureau of Standards, Washington, D. C.

* This is a Tentative Standard only, published by the American Society for Testing Materials for the purpose of eliciting criticism and suggestions. It is not a Standard of the Society and until its adoption as Standard, it is subject to revision. Issued, 1922.

† This determination of the percentage of silt, clay, loam, etc., will include all water-soluble material present, the percentage of which may be determined separately if desired.

APPENDIX X

STANDARD METHOD OF TEST FOR ORGANIC IMPURITIES IN SANDS FOR CONCRETE*

Serial Designation: C 40-22

1.—*Scope.*—The test herein specified is an approximate test for the presence of injurious organic compounds in natural sands for cement mortar or concrete. The principal value of the test is in furnishing a warning that further tests of the sand are necessary before they be used in concrete. Sands which produce a color in the sodium hydroxide solution darker than the standard color should be subjected to strength tests in mortar or concrete before use.

2.—*Sample.*—

(*a*) A representative test sample of sand of about 1 lb. shall be obtained by quartering or by the use of a sampler.

(*b*)—*Procedure.*—A 12-oz. graduated glass prescription bottle shall be filled to the 4½-oz. mark with the sand to be tested.

(*c*) A 3% solution of sodium hydroxide (NaOH) in water shall be added until the volume of sand and liquid after shaking gives a total value of 7 liquid ounces.

(*d*) The bottle shall be stoppered and shaken thoroughly and then allowed to stand for 24 hours.

(*e*) A standard color solution shall be prepared by adding 2.5 cu. cm. of a 2% solution of tannic acid in 10% alcohol to 22.5 cu. cm. of a 3% sodium hydroxide solution. This shall be placed in a 12-oz. prescription bottle, stoppered, and allowed to stand for 24 hours, then 25 cu. cm. of water added.

(*f*)—*Color Value.*—The color of the clear liquid above the sand shall be compared with the standard color solution prepared as in Paragraph (*e*) or with a glass of color similar to the standard solution.

3.—Solutions darker in color than the standard color have a "color value" higher than 250 parts per million in terms of tannic acid.

* This method is issued by the American Society for Testing Materials, under the fixed designation C 40; the final number indicates the year of original adoption as standard, or in the case of revision, the year of last revision. Proposed as tentative, 1921; adopted, 1922.

APPENDIX XI

TENTATIVE METHOD OF TEST FOR CONSISTENCY OF PORTLAND-CEMENT CONCRETE FOR PAVEMENTS OR FOR PAVEMENT BASE*

Serial Designation: D 138-22 T

1.—*Scope.*—This test covers the method to be used both in the laboratory and in the field for determining consistency of concrete† to be used for concrete pavements or for concrete base for pavements.

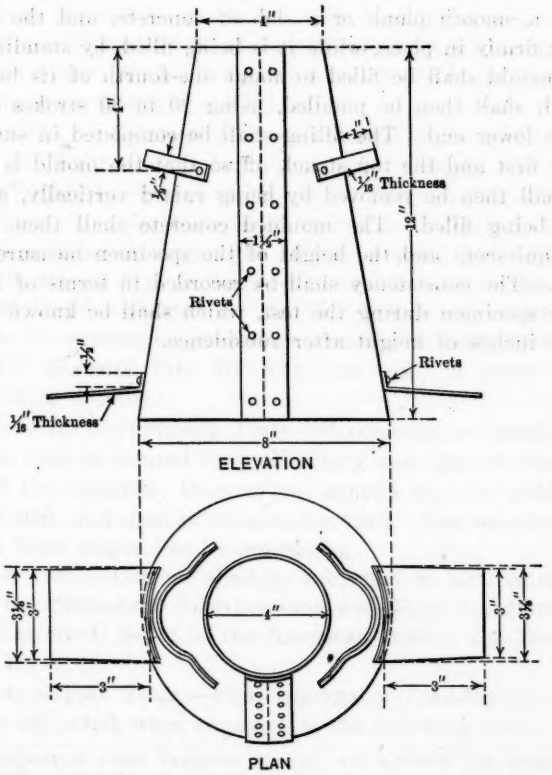


FIG. 27.

NOTE.—Criticisms of this Tentative Method are solicited and should be directed to Mr. Prévost Hubbard, Secretary of Committee D-4 on Road and Paving Materials, 25 W. Forty-third St., New York, N. Y.

* This is a Tentative Standard only, published by the American Society for Testing Materials, for the purpose of eliciting criticism and suggestions. It is not a Standard of the Society and until its adoption as Standard it is subject to revision. Issued, 1922.

† This test is not considered applicable when there is a considerable quantity of coarse aggregate more than 2 in. in size in the concrete. The Committee is now working on a method suitable for determining the consistency of concrete using aggregate more than 2 in. in size.

2.—*Apparatus.*—The test specimen shall be formed in a mould of No. 16 gauge galvanized metal in the form of the lateral surface of the frustrum of a cone with the base, 8 in. in diameter, the upper surface, 4 in. in diameter, and the altitude, 12 in. The base and the top shall be open and parallel to each other and at right angles to the axis of the cone. The mould shall be provided with foot pieces and handles as shown in Fig. 27.

3.—*Sample.*—When the test is made at the mixer, the sample shall be taken from the pile of concrete immediately after the entire batch has been discharged. When testing concrete that has been hauled from a central mixing plant, the sample shall be taken from the concrete immediately after it has been dumped on the sub-grade.

4.—*Procedure.*—The mould shall be placed on a flat, non-absorbent surface, such as a smooth plank or a slab of concrete, and the operator shall hold the form firmly in place, while it is being filled, by standing on the foot pieces. The mould shall be filled to about one-fourth of its height with the concrete which shall then be puddled, using 20 to 30 strokes of a ½-in. rod pointed at the lower end. The filling shall be completed in successive layers similar to the first and the top struck off so that the mould is exactly filled. The mould shall then be removed by being raised vertically, and at exactly 3 min. after being filled. The moulded concrete shall then be allowed to subside until quiescent and the height of the specimen measured.

5.—*Slump.*—The consistency shall be recorded in terms of inches of subsidence of the specimen during the test, which shall be known as the slump. Slump = 12 — inches of height after subsidence.

APPENDIX XII

TENTATIVE METHODS OF MAKING COMPRESSION TESTS OF CONCRETE*

Serial Designation: C 39-21 T

1.—*Scope.*—These methods are intended to cover compression tests of concrete made in a laboratory where accurate control of quantities of materials and test conditions is possible. They are designed to apply primarily to hand-mixed concrete. These methods may be used with slight modification in making tests of concrete for wearing resistance, bond between concrete and steel, impermeability, etc. The investigation of machine-mixed concrete will require certain obvious changes in the method. For methods of conducting compression tests of concrete specimens made during the progress of construction work, see the "Standard Methods of Making and Storing Specimens of Concrete in the Field" (Serial Designation: C 31–21) of the American Society for Testing Materials (Appendix XIII).

2.—*Preparation of Materials.*—Materials shall be brought to room temperatures (65 to 75° Fahr.) before beginning tests. Cement shall be stored in a dry place; preferably in covered metal cans. The cement shall be thoroughly mixed in advance, in order that the sample may be uniform throughout the tests. It shall be sifted through a No. 16 sieve and all lumps rejected. Aggregates shall be in a room-dry condition when used in concrete tests. In general, aggregates should be separated on the No. 4, ¾-in. and 1½-in. sieves† and re-combined to the average original sieve analysis for each batch. Fine aggregate should be separated into different sizes also, in cases where unusual gradings are being studied.

3.—*Sampling for Preliminary Tests.*—Representative samples‡ of all concrete materials shall be secured for preliminary tests prior to the proportioning and mixing of the concrete. Cement test samples may be made up of a small quantity from each sack used in the concrete tests. Test samples of aggregates may be taken from larger lots by quartering.

4.—*Cement Tests.*—Cement shall be subjected to test, using the methods described in the "Standard Specifications and Tests for Portland Cement" (Serial Designation: C 9–21) of the American Society for Testing Materials (Appendix II).

5.—*Fine Aggregate Tests.*—Fine aggregates (passing through a No. 4 sieve) shall be subjected, when required, to the following tests:

NOTE.—Criticisms of these Tentative Methods are solicited and should be directed to Mr. J. C. Pearson, Secretary of Committee C-9 on Concrete and Concrete Aggregates, Bureau of Standards, Washington, D. C.

* This is a Tentative Standard only, published by the American Society for Testing Materials for the purpose of eliciting criticism and suggestions. It is not a Standard of the Society and until its adoption as Standard it is subject to revision. Issued, 1921.

† For specifications for sieves, see "Standard Method of Test for Sieve Analysis of Aggregates for Concrete" (Serial Designation: C 41) of the American Society for Testing Materials, A. S. T. M. Standards, adopted in 1922 (Appendix VIII).

‡ For methods of sampling large lots of deposits of aggregate, see the "Standard Methods of Sampling Stone, Slag, Gravel, Sand, etc. for Use as Highway Materials" (Serial Designation: D 75) of the American Society for Testing Materials, A. S. T. M. Standards, adopted in 1922.

(*a*) Sieve analysis test made in accordance with the "Standard Method of Test for Sieve Analysis of Aggregates for Concrete" (Serial Designation: C 41-24) of the American Society for Testing Materials (Appendix VIII).

(*b*) Test for organic impurities (natural sand only) made in accordance with the "Standard Method of Test for Organic Impurities in Sands for Concrete" (Serial Designation: C 40-22) of the American Society for Testing Materials (Appendix X).

(*c*) Test for quantity of silt, clay or dust made in accordance with the "Tentative Method of Decantation Test for Sand and Other Fine Aggregates" (Serial Designation: D 136-22T) of the American Society for Testing Materials (Appendix IX).

(*d*) Test for unit weight made in accordance with the "Standard Method of Test for Unit Weight of Aggregate for Concrete" (Serial Designation: C 29-21) of the American Society for Testing Materials (Appendix XV).

(*e*) Strength test of 1 : 3 mortar by weight at 7 and 28 days in comparison with standard sand mortar.

6.—*Coarse Aggregate Tests.*—Coarse aggregates (retained on a No. 4 sieve) shall be subjected when required to the following tests:

(*a*) Sieve analysis test as specified under Section 5 (*a*);

(*b*) Test for quantity of silt, clay or dust, as specified under Section 5 (*c*);

(*c*) Test for unit weight as specified under Section 5 (*d*).

7.—*Mixed Aggregate Tests.*—The unit weight of mixed fine and coarse aggregates as used in concrete tests shall be determined in accordance with the method specified in Section 5 (*d*).

8.—*Proportioning.*—The quantities of each size of aggregate to be used in each batch shall be determined on the basis of the sieve analysis and the unit weight of the mixed aggregate. The exact quantities of cement and of each size of aggregate for each batch shall be determined by weight. The quantity of water for each batch shall be accurately measured. The quantities of materials may be expressed as (*a*) 1 volume of cement to . . . . volumes of total aggregate mixed as used; or (*b*) 1 volume of cement, . . . volumes fine aggregate, and . . . volumes of coarse aggregate.*

9.—*Size of Test Pieces.*—Compression tests of concrete shall be made on cylinders of a diameter equal to one-half the length. The standard shall be 6 by 12-in. cylinders where the coarse aggregate does not exceed 2 in. in size; for aggregates larger than 2 in., 8 by 16-in. cylinders shall be used; 2 by 4-in. cylinders may be used for mixtures without coarse aggregate.

10.—*Mixing Concrete.*—

(*a*) Concrete shall be mixed by hand in batches of such size as to leave a small quantity of concrete after moulding a single test piece. The batch shall preferably be mixed in a shallow galvanized steel pan with a 10-in.

* It is impracticable to give a general method for proportioning concrete for experimental purposes; the details will necessarily vary widely with the purpose for which the tests are made. The following procedure is suggested for specific cases: (*a*) Vary the cement content by 10% intervals above and below the assumed quantity; (*b*) vary the proportions of fine to coarse aggregate, measured separately, at intervals of 10%; (*c*) vary the quantity of mixing water by intervals of 10 per cent.

bricklayer's trowel which has been blunted by cutting off about 2½ in. of the point, as follows:

(*b*) The cement and fine aggregate shall be mixed dry until the mixture is homogeneous in color;

(*c*) The coarse aggregate shall be added and mixed dry;

(*d*) Sufficient water shall be added to produce concrete of the required workability;*

(*e*) The whole shall be mixed thoroughly until the entire mass is homogeneous in appearance.

11.—*Workability.*—The workability or plasticity of each batch of concrete shall be measured immediately after mixing by one of the following methods:

(*a*) Slump test made in accordance with the "Tentative Method of Test for Consistency of Portland Cement Concrete for Pavements or for Pavement Base" (Serial Designation: D 138–22T) of the American Society for Testing Materials (Appendix XI).

(*b*) Flow test made by placing a metal form in the shape of a frustum of a cone, 6¾ in. in top diameter, 10 in. in bottom diameter, 5 in. deep, on the table of the flow apparatus.† The fresh concrete shall be placed in the mould in two layers. Each layer shall be puddled and finished as described in Section 13. Immediately after moulding, the form shall be removed by a steady upward pull, the specimen raised ½ in., and dropped 15 times in about 6 sec. by means of a suitable cam and crank. The spread of the fresh concrete due to this treatment as compared with the original bottom diameter of the cone, expressed as a percentage, is the "flow."

12.—*Forms.*—The forms shall preferably be of metal. Each form shall be provided with a machined metal base-plate, and shall be oiled with a heavy mineral oil before using. Particular care shall be taken to obtain tight forms so that the mixing water will not escape during moulding.‡

13.—*Moulding Test Pieces.*—Concrete test pieces shall be moulded by placing the fresh concrete in the form in layers 3 to 4 in. in thickness. Each layer shall be puddled with twenty-five strokes with a ⅝-in. round steel bar of a length 9 in. greater than the length of the mould, pointed at the lower end. After the top layer has been puddled, the surplus concrete shall be cleaned off with a trowel, and the mould covered with a machined metal plate or a piece of plate glass at least ¼ in. thick, which will be used later in capping the test piece.

14.—*Capping Cylinders.*—Two to four hours after moulding, the test pieces shall be capped with a thin layer of stiff neat cement paste in order that the cylinders may present a smooth end for loading. The cap shall be formed by

* Concrete tests should be made on plastic mixtures. It is of the utmost importance that a uniform degree of workability be secured in tests involving comparisons of different materials and methods.

† For a description and illustration of one design for a flow-table, see *Proceedings*, Am. Soc. for Testing Materials, Vol. XX, Part II, p. 242 (1920), and *Concrete*, June, 1920, p. 274.

‡ The best type of form consists of lengths of cold-drawn steel tubing, split along one element and closed by means of a circumferential band and bolt. Satisfactory forms can be made from lengths of steel water-pipe machined on the inside, from rolled metal plates, from galvanized steel, machined iron, or steel castings. Paraffined cardboard moulds will give good results under expert supervision.

means of a machined metal plate or a piece of plate glass of suitable size, at least ¼ in. thick, worked down on the fresh cement paste until it rests on the top of the cylinder form. The cement for capping shall be mixed to a stiff paste before beginning to mix the concrete; in this way the tendency of the cap to shrink will be largely eliminated. The adhesion of the concrete to the metal base-plate and the glass can be largely eliminated by oiling the cover-plate and by inserting a sheet of paraffined tissue paper.

15.—*Curing Test Pieces.*—Concrete test pieces shall be removed from the forms 20 to 48 hours after moulding, marked, weighed, and stored in damp sand, under damp cloths or in a moist chamber until the date of test. The temperature of the curing room should not fall outside the range of 60 to 75° Fahr.

16.—*Age at Test.*—Tests shall be made at the age of 7 and 28 days; ages of 3 months and 1 year are recommended, if longer time tests are required.

17.—*Sequence of Tests.*—Three to five test pieces should be made on different days in investigations in which accurate comparisons are desired.

18.—*Preparation of Tests.*—Compression tests shall be made immediately upon removal of the concrete test pieces from the curing room; that is, the test pieces shall be loaded in a damp condition. The length and average diameter of the test piece shall be measured in inches and hundredths; two diameters shall be measured at right angles near the mid-length. The test piece shall be weighed immediately before testing.

19.—*Method of Testing.*—In general, only the ultimate compressive strength of the cylinders need be observed. The metal bearing-plates of the testing machine shall be placed in contact with the ends of the test piece; cushioning materials shall not be used. An adjustable bearing-block shall be used to transmit the load to the test piece. The bearing-block shall be placed on top of the test piece in vertical testing machines. The diameter of the bearing-block shall be approximately the same as that of the test piece. The upper section of the bearing-block shall be kept in motion as the head of the testing machine is brought to a bearing on the test piece.

20.—*Application of Load.*—The load shall be applied uniformly and without shock. The moving head of the testing machine should travel at the rate of about 0.05 in. per min. when the machine is running idle.

21.—*Record of Tests.*—The total load indicated by the testing machine at failure of the test piece shall be recorded and the unit compressive strength calculated in pounds per square inch, the area computed from the average diameter of the cylinder being used. The type of failure and appearance of the concrete shall be noted.

22.—*Weight of Concrete.*—The weight of the concrete in pounds per cubic foot shall be determined from the weight of the specimens and their dimensions.

23.—*Density and Yield.*—Density and yield of concrete when required shall be calculated from the unit volumes of the constituent materials and the volume of the concrete. Density is here understood to be the ratio of solids in the concrete to the total volume of the mass. Yield is the volume of concrete resulting from one volume of aggregate mixed as used.

24.—*Report of Tests.*—The report of tests shall include the following:

(*a*) The kind and origin of concrete materials;

(*b*) Complete data on all tests of cement and aggregates;

(*c*) A description of methods of making and testing the concrete, where methods deviate from the proposed standards;

(*d*) The quantities of cement, aggregates, and water in each batch;

(*e*) The method of measuring workability or plasticity with "slump" or "flow" of concrete;

(*f*) The quantity of water expressed as a ratio to volume of cement;

(*g*) The age at test;

(*h*) The size of test pieces;

(*i*) The date of moulding and testing each cylinder;

(*j*) The compressive strength, in pounds per square inch, of each test piece and average of tests in a set;

(*k*) A description of failure and appearance of concrete on each test piece;

(*l*) The unit weight, density, and yield of the concrete.

APPENDIX XIII

STANDARD METHODS OF MAKING AND STORING SPECIMENS OF CONCRETE IN THE FIELD*

Serial Designation C 31-21.

1.—*Scope.*—The methods herein specified apply to moulding and storing of test specimens of concrete sampled from concrete being used in construction.

2.—*Size and Shape of Specimen.*—The test specimens shall be cylindrical in form with the length twice the diameter. In general a mould whose diameter is not less than four times the diameter of the largest size aggregate shall be used. (The sizes most commonly used are 6 by 12-in. and 8 by 16-in. cylinders.)

3.—*Moulds.*—

(*a*) The moulds shall be cylindrical in form, made of non-absorbent material, and shall be substantial enough to hold their form during the moulding of the test specimens. They shall not vary in diameter more than $\frac{1}{8}$ in. in any direction, nor shall they vary in height more than $\frac{1}{8}$ in. from the height required. They shall be substantially water-tight so that there will be no leakage of water from the test specimen during moulding.

(*b*) Each mould shall be provided with a base-plate having a plane surface and made of non-absorbent material. This plate shall be large enough in diameter to support properly the form without leakage. Plate-glass or planed metal are satisfactory for this purpose. A similar plate should be provided for covering the top surface of the test specimen after being moulded.

(*c*) Suggestions for suitable forms are shown in Figs. 28, 29 and 30.

4.—*Sampling of Concrete.*—

(*a*) Concrete for the test specimens shall be taken immediately after it has been placed in the work. All the concrete for each sample shall be taken from one place. A sufficient number of samples—each large enough to make one test specimen—shall be taken at different points so that the test specimens made from them will give a fair average of the concrete placed in that portion of the structure selected for tests. The location from which each sample is taken shall be noted clearly for future reference.

(*b*) In securing samples, the concrete shall be taken from the mass by a shovel or similar implement and placed in a large pail or other receptacle, for transporting to the point of moulding. Care shall be taken to see that each test specimen represents the total mixture of the concrete at that place. Different samples shall not be mixed together, but each sample shall make one specimen.

* These methods are issued by the American Society for Testing Materials, under the fixed designation C 31; the final number indicates the year of original adoption as standard, or in the case of revision, the year of last revision. Proposed as tentative, 1920; adopted, 1921.

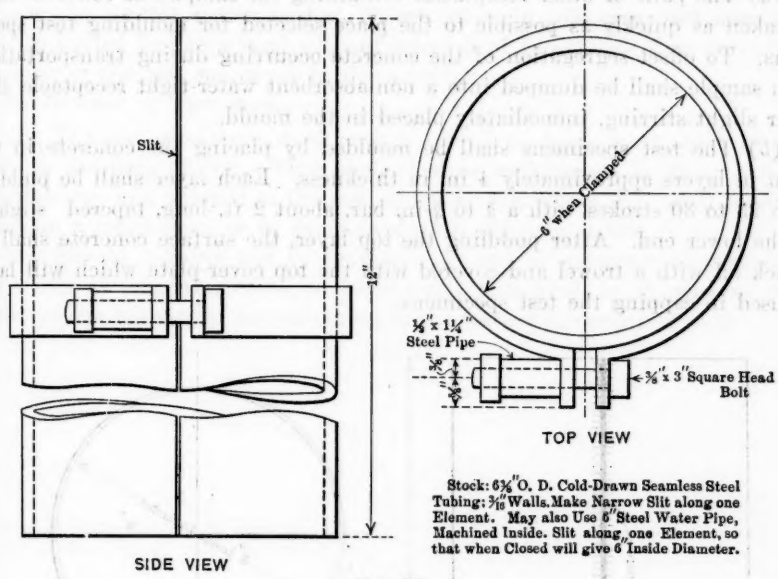

Slit
12"
SIDE VIEW
6" when Clamped
⅛"x 1¼" Steel Pipe
⅜"x 3" Square Head Bolt
TOP VIEW
Stock: 6⅛" O. D. Cold-Drawn Seamless Steel Tubing; 3/16" Walls. Make Narrow Slit along one Element. May also Use 6" Steel Water Pipe, Machined Inside. Slit along one Element, so that when Closed will give 6" Inside Diameter.

FIG. 28.

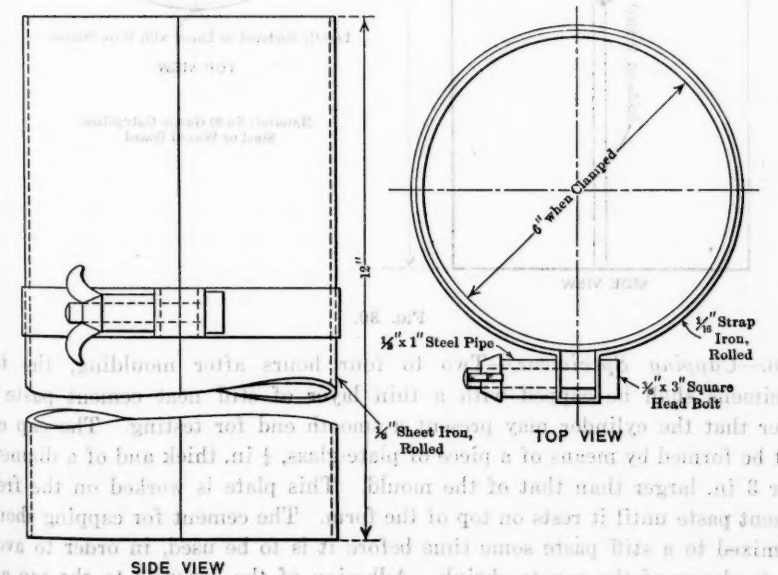

12"
SIDE VIEW
6" when Clamped
1/16" Strap Iron, Rolled
⅛"x 1" Steel Pipe
⅜"x 3" Square Head Bolt
⅛" Sheet Iron, Rolled
TOP VIEW

FIG. 29.

5.—*Moulding the Specimens.*—

(*a*) The pails or other receptacles containing the samples of concrete shall be taken as quickly as possible to the place selected for moulding test specimens. To offset segregation of the concrete occurring during transportation, each sample shall be dumped into a non-absorbent water-tight receptacle and, after slight stirring, immediately placed in the mould.

(*b*) The test specimens shall be moulded by placing the concrete in the form in layers approximately 4 in. in thickness. Each layer shall be puddled with 25 to 30 strokes with a $\frac{5}{8}$ to $\frac{3}{4}$-in. bar, about 2 ft. long, tapered slightly at the lower end. After puddling the top layer, the surface concrete shall be struck off with a trowel and covered with the top cover-plate which will later be used in capping the test specimens.

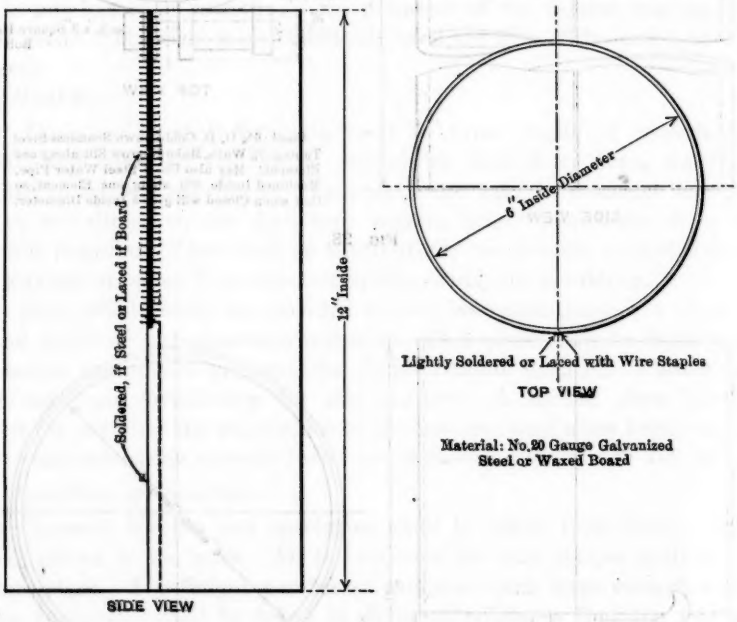

Soldered, if Steel or Laced if Board
12" Inside
SIDE VIEW
6" Inside Diameter
Lightly Soldered or Laced with Wire Staples
TOP VIEW
Material: No. 20 Gauge Galvanized Steel or Waxed Board

FIG. 30.

6.—*Capping Specimens.*—Two to four hours after moulding, the test specimens shall be capped with a thin layer of stiff neat cement paste in order that the cylinder may present a smooth end for testing. The cap can best be formed by means of a piece of plate-glass, $\frac{1}{4}$ in. thick and of a diameter 2 or 3 in. larger than that of the mould. This plate is worked on the fresh cement paste until it rests on top of the form. The cement for capping should be mixed to a stiff paste some time before it is to be used, in order to avoid the tendency of the cap to shrink. Adhesion of the concrete to the top and bottom plates can be avoided by oiling the plates or by inserting a sheet of paraffined tissue paper.

7.—*Removal of Specimens from Forms.*—At the end of 48 hours the test specimens shall be removed from the moulds and buried in damp sand, except in case the moulds shown in Fig. 30 are used; in this case, test specimens may be buried in damp sand without removal of the mould, thus permitting shipping of the test specimens in the moulds.

8.—*Storage of Specimens.*—

(*a*) The test specimens shall remain buried in damp sand until 10 days prior to the date of test. They shall then be well packed in damp sand or wet shavings and shipped to the testing laboratory, where they shall be stored either in a moist room or in damp sand until the date of test.

(*b*) Should a 7-day test be required, the test specimens shall remain at the works as long as possible to harden and then shall be shipped so as to arrive at the laboratory in time to make the test on the required date.

APPENDIX XIV

STANDARD SPECIFICATIONS FOR CAST-IRON PIPE AND SPECIAL CASTINGS*

Serial Designation: A 44-04.

Description of Pipes

1.—The pipes shall be made with hub-and-spigot joints, and shall accurately conform to the dimensions given in Tables 20 and 21. They shall be straight and shall be true circles in section, with their inner and outer surfaces concentric, and shall be of the specified dimensions in outside diameter. They shall be at least 12 ft. in length, exclusive of socket. For pipes of each size from 4 in. to 24 in., inclusive, there shall be two standards of outside diameter, and for pipes from 30 in. to 60 in., inclusive, there shall be four standards of outside diameter, as shown by Table 21.

All pipes having the same outside diameter shall have the same inside diameter at both ends. The inside diameter of the lighter pipes of each standard outside diameter shall be gradually increased for a distance of about 6 in. from each end of the pipe so as to obtain the required standard thickness and weight for each size and class of pipe.

Pipes whose standard thickness and weight are intermediate between the classes in Table 21, shall be made of the same outside diameter as the next heavier class. Pipes whose standard thickness and weight are less than shown by Table 21 shall be made of the same outside diameter as the Class A pipes, and pipes whose thickness and weight are more than shown by Table 21 shall be made of the same outside diameter as the Class D pipes.

For pipes 4 in. to 12 in., inclusive, one class of special castings shall be furnished, made from Class D pattern. Those having spigot ends shall have outside diameters of spigot ends midway between the two standards of outside diameter as shown by Table 21, and shall be tapered back for a distance of 6 in. For pipes from 14 in. to 24 in., inclusive, two classes of special castings shall be furnished, Class B special castings with Classes A and B pipes, and Class D special castings with Classes C and D pipes, the former to be stamped "AB" and the latter to be stamped "CD." For pipes 30 in. to 60 in., inclusive, four classes of special castings shall be furnished, one for each class of pipe, and shall be stamped with the letter of the class to which they belong.

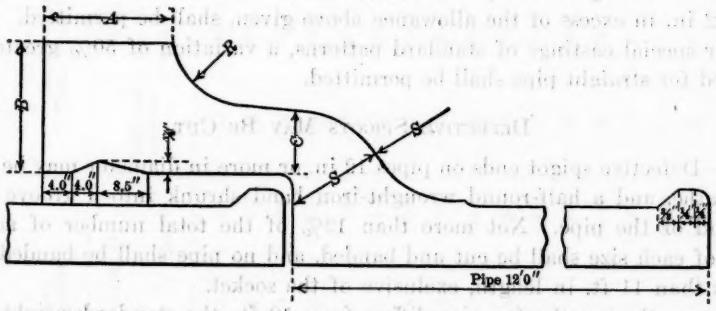
Allowable Variation in Diameter of Pipes and Sockets

2.—Especial care shall be taken to have the sockets of the required size. The sockets and spigots will be tested by circular gauges, and no pipe will be received which is defective in joint room from any cause. The diameters of the sockets and the outside diameters of the bead ends of the pipes shall not

* These specifications are issued by the American Society for Testing Materials, under the fixed designation A 44; the final number indicates the year of original adoption as standard, or in the case of revision, the year of last revision. Adopted, 1904.

vary from the standard dimensions by more than 0.06 in. for pipes 16 in. or less in diameter; 0.08 in. for 18-in., 20-in., and 24-in. pipes; 0.10 in. for 30-in., 36-in., and 42-in. pipes; 0.12 in. for 48-in. pipes; and 0.15 in. for 54-in. and 60-in. pipes.

TABLE 20.—GENERAL DIMENSIONS OF PIPES



Nominal diameter, in inches.	Classes.	Actual outside diameter, in inches.	Diameter of Sockets.		Depth of Sockets.		A	B	C
			Pipe, in inches.	Special castings, in inches.	Pipe, in inches.	Special castings, in inches.			
4	A—B	4.80	5.60	5.70	3.50	4.00	1.5	1.30	0.65
4	C—D	5.00	5.80	5.70	3.50	4.00	1.5	1.30	0.65
6	A—B	6.90	7.70	7.80	3.50	4.00	1.5	1.40	0.70
6	C—D	7.10	7.90	7.80	3.50	4.00	1.5	1.40	0.70
8		9.05	9.85	10.00	4.00	4.00	1.5	1.50	0.75
8	C—D	9.30	10.10	10.00	4.00	4.00	1.5	1.50	0.75
10	A—B	11.10	11.90	12.10	4.00	4.00	1.5	1.50	0.75
10	C—D	11.40	12.20	12.10	4.00	4.00	1.5	1.60	0.80
12	A—B	13.20	14.00	14.20	4.00	4.00	1.5	1.60	0.80
12	C—D	13.50	14.30	14.20	4.00	4.00	1.5	1.70	0.85
14	A—B	15.30	16.10	16.10	4.00	4.00	1.5	1.70	0.85
14	C—D	15.65	16.45	16.45	4.00	4.00	1.5	1.80	0.90
16	A—B	17.40	18.40	18.40	4.00	4.00	1.75	1.80	0.90
16	C—	17.80	18.80	18.80	4.00	4.00	1.75	1.90	1.00
18	A—B	19.50	20.50	20.50	4.00	4.00	1.75	1.90	0.95
18	C—D	19.92	20.92	20.92	4.00	4.00	1.75	2.10	1.05
20	A—B	21.60	22.60	22.60	4.00	4.00	1.75	2.00	1.00
20	C—D	22.06	23.06	23.06	4.00	4.00	1.75	2.30	1.15
24	A—B	25.80	26.80	26.80	4.00	4.00	2.00	2.10	1.05
24	C—D	26.32	27.32	27.32	4.00	4.00	2.00	2.50	1.25
30	A	31.74	32.74	32.74	4.50	4.50	2.00	2.50	1.15
30	B	32.00	33.00	33.00	4.50	4.50	2.00	2.30	1.15
30	C	32.40	33.40	33.40	4.50	4.50	2.00	2.60	1.32
30	D	32.74	33.74	33.74	4.50	4.50	2.00	3.00	1.50
36	A	37.96	38.96	38.96	4.50	4.50	2.00	2.50	1.25
36	B	38.30	39.30	39.30	4.50	4.50	2.00	2.80	1.40
36	C	38.70	39.70	39.70	4.50	4.50	2.00	3.10	1.60
36	D	39.16	40.16	40.16	4.50	4.50	2.00	3.40	1.80
42	A	44.20	45.20	45.20	5.00	5.00	2.00	2.80	1.40
42	B	44.55	45.50	45.50	5.00	5.00	2.00	3.00	1.50
42	C	45.10	46.10	46.10	5.00	5.00	2.00	3.40	1.75
42	D	45.58	46.58	46.58	5.00	5.00	2.00	3.80	1.95
48	A	50.50	51.50	51.50	5.00	5.00	2.00	3.00	1.50
48	B	50.80	51.80	51.80	5.00	5.00	2.00	3.30	1.65
48	C	51.40	52.40	52.40	5.00	5.0	2.00	3.80	1.95
48	D	51.98	52.98	52.98	5.00	5.0	2.00	4.20	2.20
54	A	56.66	57.66	57.66	5.50	5.5	2.25	3.20	1.60
54	B	57.10	58.10	58.10	5.50	5.5	2.25	3.60	1.80
54	C	57.80	58.80	58.80	5.50	5.50	2.25	4.40	2.15
54	D	58.40	59.40	59.40	5.50	5.50	2.25	4.40	2.45
60	A	62.80	63.80	63.80	5.50	5.50	2.25	3.40	1.70
60	B	63.40	64.40	64.40	5.50	5.50	2.25	3.70	1.90
60	C	64.20	65.20	65.20	5.50	5.50	2.25	4.20	2.25
60	D	64.82	65.82	65.82	5.50	5.50	2.25	4.70	2.60

Allowable Variation in Thickness

3.—For pipes whose standard thickness is less than 1 in., the thickness of metal in the body of the pipe shall not be more than 0.08 in. less than the standard thickness; and for pipes whose standard thickness is 1 in. or more, the variation shall not exceed 0.10 in., except that for spaces not exceeding 8 in. in length in any direction, variations from the standard thickness of 0.02 in. in excess of the allowance above given, shall be permitted.

For special castings of standard patterns, a variation of 50% greater than allowed for straight pipe shall be permitted.

Defective Spigots May Be Cut

4.—Defective spigot ends on pipes 12 in. or more in diameter may be cut off in a lathe, and a half-round wrought-iron band shrunk into a groove cut in the end of the pipe. Not more than 12% of the total number of accepted pipes of each size shall be cut and banded, and no pipe shall be banded which is less than 11 ft. in length, exclusive of the socket.

In case the length of a pipe differs from 12 ft., the standard weight of the pipe given in Table 21 shall be modified in accordance therewith.

Special Castings

5.—All special castings shall be made in accordance with the cuts and the dimensions given in the table forming a part of these specifications.

The diameters of the sockets and the external diameters of the bead ends of the special castings shall not vary from the standard dimensions by more than 0.12 in. for castings 16 in. or less in diameter; 0.15 in. for 18-in., 20-in., and 24-in. pipes; 0.20 in. for 30-in., 36-in., and 42-in. pipes; and 0.24 in. for 48-in., 54-in., and 60-in. pipes. These variations apply only to special castings made from standard patterns.

The flanges on all manhole castings and manhole covers shall be faced true and smooth, and drilled to receive bolts of the sizes given in the tables. The manufacturer shall furnish and deliver all bolts for bolting on the manhole covers, the bolts to be of the sizes shown on plans and made of the best quality of mild steel, with hexagonal heads and nuts and sound, well-fitting threads.

Marking

6.—Every pipe and special casting shall have distinctly cast upon it the initials of the maker's name. When cast especially to order, each pipe and special casting larger than 4 in. may also have cast upon it figures showing the year in which it was cast and a number signifying the order in point of time in which it was cast, the figures denoting the year being above and the number below, thus:

1901	1901	1901
1	2	3

etc., also any initials, not exceeding four, which may be required by the purchaser. The letters and figures shall be cast on the outside and shall be

TABLE 21.—STANDARD THICKNESSES AND WEIGHTS OF CAST-IRON PIPE*

Nominal inside diameter, in inches.	Class A. 100-ft. head. 43-lb. pressure.			Class B. 200-ft. head. 86-lb. pressure.			Class C. 300-ft. head. 130-lb. pressure.			Class D. 400-ft. head. 173-lb. pressure.			Nominal inside diameter, in inches.
	Thickness, in inches.	Weight per		Thickness, in inches.	Weight per		Thickness, in inches.	Weight per		Thickness, in inches.	Weight per		
		Foot.	Length.		Foot.	Length.		Foot.	Length,		Foot.	Length.	
4	0.42	20.0	240	0.45	21.7	260	0.48	23.3	280	0.52	25.0	300	4
6	0.44	30.8	370	0.48	33.3	400	0.51	35.8	430	0.55	38.3	460	6
8	0.46	42.9	515	0.51	47.5	570	0.56	52.1	625	0.60	55.8	670	8
10	0.50	57.1	685	0.57	63.8	765	0.62	70 8	850	0.68	76.7	920	10
12	0.54	72.5	870	0.62	82.1	985	0.68	91.7	1 100	0.75	100.0	1 200	12
14	0.57	89.6	1 075	0.66	102.5	1 230	0.74	116.7	1 400	0.82	129.2	1 550	14
16	0.60	108.3	1 300	0.70	125.0	1 500	0.80	143.8	1 725	0.89	158.3	1 900	16
18	0.64	129.2	1 550	0.75	150.0	1 800	0.87	175.0	2 100	0.96	191.7	2 300	18
20	0.67	150.0	1 800	0.80	175.0	2 100	0.92	208.3	2 500	1.03	229.2	2 750	20
24	0.76	204.2	2 450	0.89	233.3	2 800	1.04	279.2	3 350	1.16	306.7	3 680	24
30	0.88	291.7	3 500	1.03	333.3	4 000	1.20	400.0	4 800	1.37	450.0	5 400	30
36	0.99	391.7	4 700	1.15	454.2	5 450	1.36	545.8	6 550	1.58	625.0	7 500	36
42	1.10	512.5	6 150	1.28	591.7	7 100	1.54	716.7	8 600	1.78	825.0	9 900	42
48	1.26	666.7	8 000	1.42	750.0	9 000	1.71	908.3	10 900	1.96	1 050.0	12 600	48
54	1.35	800.0	9 600	1.55	933.3	11 200	1.90	1 141.7	13 700	2.23	341.7	16 100	54
60	1.39	916.7	11 000	1.67	1 104.2	13 250	2.00	1 341.7	16 100	2.38	1 583.3	19 000	60

* The above weights are for 12-ft. laying lengths and standard sockets; proportionate allowance to be made for any variation therefrom.

not less than 2 in. in length and ⅛ in. in relief for pipes 8 in. in diameter and larger. For smaller sizes of pipes, the letters may be 1 in. in length. The weight and the class letter shall be conspicuously painted in white on the inside of each pipe and special casting after the coating has become hard.

Allowable Percentage of Variation in Weight

7.—No pipe shall be accepted the weight of which shall be less than the standard weight by more than 5% for pipes 16 in. or less in diameter, and 4% for pipes more than 16 in. in diameter; and no excess above the standard weight of more than the given percentages for the several sizes shall be paid for. The total weight to be paid for shall not exceed for each size and class of pipe received the sum of the standard weights of the same number of pieces of the given size and class by more than 2 per cent.

No special casting shall be accepted the weight of which shall be less than the standard weight by more than 10% for pipes 12 in. or less in diameter, and 8% for larger sizes, except that curves, Y-pieces, and breeches pipe may be 12% below the standard weight, and no excess above the standard weight of more than the above percentages for the several sizes will be paid for. These variations apply only to castings made from the standard patterns.

Quality of Iron

8.—All pipes and special castings shall be made of cast iron of good quality, and of such character as shall make the metal of the castings strong, tough, and of even grain, and soft enough to admit satisfactorily of drilling and cutting. The metal shall be made without any admixture of cinder iron or other inferior metal, and shall be re-melted in a cupola or air furnace.

Tests of Materials

9.—Specimen bars of the metal used, each being 26 in. long by 2 in. wide, and 1 in. thick, shall be made without charge as often as the Engineer may direct, and, in default of definite instructions, the contractor shall make and test at least one bar from each heat or run of metal. The bars, when placed flat-wise on supports 24 in. apart and loaded in the center, shall, for pipes 12 in. or less in diameter, support a load of 1 900 lb. and show a deflection of not less than 0.30 in. before breaking; and for pipes of sizes larger than 12 in., they shall support a load of 2 000 lb. and show a deflection of not less than 0.32 in. The contractor shall have the right to make and break three bars from each heat or run of metal, and the test shall be based on the average results of the three bars. Should the dimensions of the bars differ from those above given, a proper allowance therefor shall be made in the results of the tests.

Casting of Pipes

10.—The straight pipes shall be cast in dry sand moulds in a vertical position. Pipes 16 in. or less in diameter shall be cast with the hub end up or down, as specified in the proposal. Pipes 18 in. or more in diameter shall be cast with the hub end down.

The pipes shall not be stripped or taken from the pit while showing color of heat, but shall be left in the flasks for a sufficient length of time to prevent unequal contraction by subsequent exposure.

Quality of Castings

11.—The pipes and special castings shall be smooth, free from scales, lumps, blisters, sand-holes, and defects of every nature which unfit them for the use for which they are intended. No plugging or filling will be allowed.

Cleaning and Inspection

12.—All pipes and special castings shall be thoroughly cleaned and subjected to a careful hammer inspection. No casting shall be coated unless entirely clean and free from rust, and approved in these respects by the Engineer immediately before being dipped.

Coating

13.—Every pipe and special casting shall be coated inside and out with coal-tar pitch varnish. The varnish shall be made from coal-tar. To this material sufficient oil shall be added to make a smooth coating, tough and tenacious when cold, and not brittle, nor with any tendency to scale off.

Each casting shall be heated to a temperature of 300° Fahr. immediately before it is dipped, and shall possess not less than this temperature at the time it is put in the vat. The ovens in which the pipes are heated shall be arranged so that all portions of the pipe shall be heated to an even temperature. Each casting shall remain in the bath at least 5 min.

The varnish shall be heated to a temperature of 300° Fahr. (or less if the Engineer shall so order), and shall be maintained at this temperature during the time the casting is immersed.

Fresh pitch and oil shall be added when necessary to keep the mixture at the proper consistency, and the vat shall be emptied of its contents and refilled with fresh pitch when deemed necessary by the Engineer. After being coated, the pipes shall be carefully drained of the surplus varnish. Any pipe or special casting that is to be re-coated shall first be thoroughly scraped and cleaned.

Hydrostatic Test

14.—When the coating has become hard, the straight pipes shall be subjected to a proof by hydrostatic pressure and, if required by the Engineer, they shall also be subjected to a hammer test under this pressure.

The pressure to which the different sizes and classes of pipes shall be subjected are as follows (Table 22).

Weighing

15.—The pipes and special castings shall be weighed for payment under the supervision of the Engineer after the application of the coal-tar pitch varnish. If desired by the Engineer the pipes and special castings shall be weighed after their delivery, and the weights so ascertained shall be used in the final settlement, provided such weighing is done by a legalized weigh-master. Bids shall be submitted and a final settlement made on the basis of a ton of 2 000 lb.

TABLE 22.

	20-in. diameter and larger, in pounds per square inch.	Less than 20-in. diameter, in pounds per square inch.
Class A pipe	150	300
Class B pipe	200	300
Class C pipe	250	300
Class D pipe	300	300

Contractor to Furnish Men and Materials

16.—The contractor shall provide all tools, testing machines, materials, and men necessary for the required testing, inspection, and weighing at the foundry of the pipes and special castings; and, should the purchaser have no inspector at the works, the contractor shall, if required by the Engineer, furnish a sworn statement that all the tests have been made as specified, this statement to contain the results of the tests upon the test-bars.

Power of Engineer to Inspect

17.—The Engineer shall be at liberty at all times to inspect the material at the foundry, and the moulding, casting, and coating of the pipes and special castings. The forms, sizes, uniformity, and conditions of all pipes and other castings herein referred to shall be subject to his inspection and approval, and he may reject, without proving, any pipes or other castings which are not in conformity with the specifications or drawings.

Inspector to Report

18.—The inspector at the foundry shall report daily to the foundry office all pipes and special castings rejected, with the causes for rejection.

Castings to Be Delivered Sound and Perfect

19.—All the pipes and other castings must be delivered in all respects sound and conformable to these specifications. The inspection shall not relieve the contractor of any of his obligations in this respect, and any defective pipe or other castings which may have passed the Engineer at the works or elsewhere shall be at all times liable to rejection when discovered until the final completion and adjustment of the contract, provided, however, that the contractor shall not be held liable for pipes or special castings found to be cracked after they have been accepted at the agreed point of delivery. Care shall be taken in handling the pipes not to injure the coating, and no pipes or other material of any kind shall be placed in the pipes during transportation or at any time after they received the coating.

Definition of the Word "Engineer"

20.—Wherever the word "Engineer" is used herein it shall be understood to refer to the Engineer or inspector acting for the purchaser and to his properly authorized agents, limited by the particular duties entrusted to them

APPENDIX XV

STANDARD METHOD OF TEST FOR UNIT WEIGHT OF AGGREGATE FOR CONCRETE*

Serial Designation: C 29–21.

1.—The unit weight of fine, coarse, or mixed aggregates for concrete shall be determined by the following method:

2.—*Apparatus.*—

(*a*) The apparatus required consists of a cylindrical metal measure, a tamping rod, and a scale or balance, sensitive to 0.5% of the weight of the sample to be weighed.

(*b*) *Measures.*—The measure shall be of metal, preferably machined to accurate dimensions on the inside, cylindrical in form, water-tight, and of sufficient rigidity to retain its form under rough usage, with top and bottom true and even, and preferably provided with handles.

The measure shall be of $\frac{1}{10}$, $\frac{1}{2}$, or 1-cu. ft. capacity, depending on the maximum diameter of the coarsest particles in the aggregate, and shall be of the following dimensions (Table 23).

TABLE 23.

Capacity, in cubic feet.	Inside diameter, in inches.	Inside height, in inches.	Minimum thickness of metal, U. S. Gauge.	Diameter of largest particles of aggregate, in inches.
$\frac{1}{10}$	6.00	6.10	No. 11	Under $\frac{1}{2}$
$\frac{1}{2}$	10.00	11.00	No. 8	Under $1\frac{1}{2}$
1	14.00	11.28	No. 5	Over $1\frac{1}{2}$

(*c*) *Tamping Rod.*—The tamping rod shall be a straight metal rod, $\frac{5}{8}$ in. in diameter and 18 in. long, with one end tapered for a distance of 1 in. to a blunt bullet-shape point.

3.—*Calibrating the Measure.*—The measure shall be calibrated by accurately determining the weight of water at 16.7° cent. (62° Fahr.) required to fill it. The factor for any unit shall be obtained by dividing the unit weight of water at 16.7° cent. (62° Fahr.)† by the weight of water at 16.7° cent. (62° Fahr.) required to fill the measure.

4.—The sample of aggregate shall be room-dry and thoroughly mixed.

5.—*Method.*—

(*a*) The measure shall be filled one-third full and the top leveled off with the fingers. The mass shall be tamped with the pointed end of the tamping rod twenty-five times, evenly distributed over the surface. The measure shall be filled two-thirds full and again tamped twenty-five times as before. The

* This method is issued by the American Society for Testing Materials under the fixed designation C 29; the final number indicates the year of original adoption as standard, or in the case of revision, the year of last revision. Proposed as tentative, 1920; adopted, 1921.

† The unit weight of water at 16.7° cent. (62° Fahr.) is 62 355 lb. per cu. ft.

measure shall then be filled to overflowing, tamped twenty-five times, and the surplus aggregate struck off, using the tamping rod as a straight edge.

In tamping the first layer, the rod should not be permitted to forcibly strike the bottom of the measure. In tamping the second and final layers, only enough force to cause the tamping rod to penetrate the last layer of aggregate placed in the measure should be used. No effort should be made to fill holes left by the rod when the aggregate is damp.

(*b*) The net weight of the aggregate in the measure shall be determined. The unit weight of the aggregate shall then be obtained by multiplying the net weight of the aggregate by the factor found as described in Section 3.

6.—*Accuracy.*—Results with the same sample should check within 1 per cent.

ADVISORY APPENDICES

APPENDIX XVI

PROPORTIONS* FOR CONCRETE OF A GIVEN COMPRESSIVE STRENGTH AT 28 DAYS

The table gives the proportions in which Portland cement and a wide range in sizes of fine and coarse aggregates should be mixed to obtain concrete of compressive strengths ranging from 1 500 to 3 000 lb. per sq. in. at 28 days. Proportions are given for concrete of four different consistencies.

The purpose of the table is twofold:

1.—To furnish a guide in the selection of mixtures to be used in preliminary investigations of the strength of concrete from given materials.

2.—To indicate proportions which may be expected to produce concrete of a given strength under average conditions where control tests are not made.

If the proportions to be used in the work are selected from the table without preliminary tests of the materials, and control tests are not made during the progress of the work, the mixtures in bold-face type shall be used.

The use of this table as a guide in the selection of concrete mixtures is based on the following:

1.—Concrete shall be plastic;

2.—Aggregates shall be clean and structurally sound;

3.—Aggregates shall be graded between the sizes indicated;

4.—Cement shall conform to the requirements of the "Standard Specifications and Tests for Portland Cement (Serial Designation: C 9–21) of the American Society for Testing Materials (Appendix II).

The plasticity of the concrete shall be determined by the slump test carried out in accordance with the "Tentative Method of Test for Consistency of Portland Cement Concrete for Pavements or for Pavement Base" (Serial Designation: D 138–22 T) of the American Society for Testing Materials (Appendix XI).

Apply the following rules in determining the size assigned to a given aggregate:

1.—Not less than 15% shall be retained between the sieve which is considered the maximum size† and the next smaller sieve.

2.—Not more than 15% of a coarse aggregate shall be finer than the sieve considered as the minimum size.†

* Based on the 28-day compressive strengths of 6 by 12-in. cylinders, made and stored in accordance with the "Tentative Methods of Making Compression Tests of Concrete" (Serial Designation: C 39-21 T) of the American Society for Testing Materials (Appendix XII).

† For example: a graded sand with 16% retained on the No. 8 sieve would fall in the 0-No. 4 size; if 14% or less were retained, the sand would fall in the 0-No. 8 size. A coarse aggregate having 16% coarser than 2-in. sieve would be considered as 3-in. aggregate.

3.—Only the sieve sizes given in the table shall be considered in applying Rules (1) and (2).

4.—Sieve analysis shall be made in accordance with the "Tentative Method of Test for Sieve Analysis of Aggregates for Concrete" (Serial Designation: C 41–24 T) of the American Society for Testing Materials (Appendix VIII).

Proportions may be interpolated for concrete strengths, aggregate sizes and consistencies not covered by the table or determined by test.

Proportions for 1 500 Lb. per Sq. In. Concrete.

Proportions are expressed by volume as follows: Portland cement; fine aggregate; coarse aggregate.

Thus 1:2.6:4.6 indicates 1 part by volume of Portland cement, 2.6 parts by volume of fine aggregate, and 4.6 parts by volume of coarse aggregate.

Size of coarse aggregate	Slump, in inches	Size of Fine Aggregate				
		0 – No. 28	0 – No. 14	0 – No. 8	0 – No. 4	0 – ⅜ in.
None	½ to 1	1:2.8	1:3.2	1:3.8	1:4.4	1:5.1
	3 " 4	1:2.4	1:2.8	1:3.3	1:3.8	1:4.5
	6 " 7	1:1.9	1:2.2	1:2.6	1:3.0	1:3.6
	8 "10	1:1.4	1:1.6	1:1.8	1:2.1	1:2.5
No. 4 to ¾ in.	½ to 1	1:2.6:4.6	1:2.9:4.3	1:3.4:4.1	1:3.9:3.6	1:4.6:3.1
	3 " 4	1:2.3:4.0	1:2.6:3.8	1:2.9:3.6	1:3.4:3.2	1:4.1:2.8
	6 " 7	1:1.8:3.4	1:2.0:3.2	1:2.3:3.1	1:2.6:2.8	1:3.1:2.5
	8 "10	1:1.1:2.5	1:1.3:2.4	1:1.5:2.4	1:1.7:2.2	1:2.1:2.0
No. 4 to 1 in.	½ to 1	1:2.4:5.3	1:2.7:5.2	1:3.1:5.0	1:3.5:4.7	1:4.3:4.3
	3 " 4	1:2.1:4.7	1:2.4:4.5	1:2.7:4.4	1:3.1:4.1	1:3.7:3.7
	6 " 7	1:1.6:3.9	1:1.8:3.8	1:2.1:3.7	1:2.4:3.5	1:2.9:3.3
	8 "10	1:1.1:2.9	1:1.2:2.8	1:1.4:2.8	1:1.6:2.7	1:1.9:2.5
No. 4 to 1½ in.	½ to 1	1:2.4:6.0	1:2.7:5.9	1:3.1:5.8	1:3.5:5.4	1:4.1:5.1
	3 " 4	1:2.0:5.4	1:2.3:5.3	1:2.7:5.2	1:3.0:5.0	1:3.5:4.6
	6 " 7	1:1.6:4.4	1:1.8:4.3	1:2.0:4.3	1:2.3:4.1	1:2.7:3.9
	8 "10	1:1.0:3.3	1:1.1:3.2	1:1.3:3.2	1:1.5:3.1	1:1.8:2.9
No. 4 to 2 in.	½ to 1	1:2.2:6.9	1:2.4:6.8	1:2.8:6.8	1:3.1:6.6	1:3.7:6.4
	3 " 4	1:1.8:6.2	1:2.0:6.1	1:2.4:6.1	1:2.7:6.0	1:3.1:5.7
	6 " 7	1:1.4:5.1	1:1.6:5.0	1:1.8:5.0	1:2.0:5.0	1:2.4:4.8
	8 "10	1:0.9:3.8	1:1.0:3.8	1:1.1:3.8	1:1.3:3.8	1:1.5:3.7
⅛ to 1 in.	½ to 1	1:2.8:5.2	1:3.1:5.1	1:3.6:4.8	1:4.2:4.6	1:4.8:4.1
	3 " 4	1:2.4:4.5	1:2.6:4.5	1:3.1:4.3	1:3.6:4.0	1:4.1:3.6
	6 " 7	1:1.9:3.9	1:2.1:3.7	1:2.4:3.6	1:2.8:3.4	1:3.2:3.1
	8 "10	1:1.3:2.8	1:1.4:2.8	1:1.6:2.7	1:1.9:2.6	1:2.2:2.4
⅛ to 1½ in.	½ to 1	1:2.8:5.8	1:3.1:5.7	1:3.5:5.5	1:4.1:5.3	1:4.7:4.9
	3 " 4	1:2.4:5.2	1:2.7:5.1	1:3.1:5.0	1:3.5:4.8	1:4.1:4.4
	6 " 7	1:1.9:4.3	1:2.1:4.2	1:2.4:4.2	1:2.7:4.0	1:3.1:3.7
	8 "10	1:1.2:3.2	1:1.4:3.2	1:1.6:3.1	1:1.8:3.0	1:2.1:2.9
⅛ to 2 in.	½ to 1	1:2.7:6.6	1:3.0:6.6	1:3.4:6.5	1:3.9:6.4	1:4.4:6.0
	3 " 4	1:2.3:5.9	1:2.6:5.9	1:2.9:5.8	1:3.3:5.6	1:3.7:5.5
	6 " 7	1:1.8:4.9	1:2.0:4.8	1:2.2:4.8	1:2.6:4.8	1:3.0:4.5
	8 "10	1:1.2:3.7	1:1.3:3.7	1:1.5:3.7	1:1.7:3.6	1:1.9:3.5
¼ to 1½ in.	½ to 1	1:3.2:5.4	1:3.6:5.3	1:4.1:5.1	1:4.7:4.8	1:5.3:4.4
	3 " 4	1:2.8:4.8	1:3.2:4.8	1:3.6:4.6	1:4.0:4.4	1:4.6:4.0
	6 " 7	1:2.1:4.0	1:2.5:4.0	1:2.8:3.9	1:3.2:3.7	1:3.5:3.4
	8 "10	1:1.5:3.0	1:1.7:3.0	1:1.9:2.9	1:2.2:2.8	1:2.5:2.7
¼ to 2 in.	½ to 1	1:3.2:6.2	1:3.6:6.1	1:4.0:6.0	1:4.6:5.8	1:5.2:5.4
	3 " 4	1:2.8:5.5	1:3.1:5.5	1:3.5:5.4	1:3.9:5.2	1:4.5:4.9
	6 " 7	1:2.1:4.5	1:2.4:4.6	1:2.7:4.5	1:3.1:4.4	1:3.5:4.1
	8 "10	1:1.4:3.4	1:1.6:3.4	1:1.8:3.4	1:2.1:3.4	1:2.4:3.3
¼ to 3 in.	½ to 1	1:3.2:7.1	1:3.6:7.1	1:4.0:7.0	1:4.6:6.9	1:5.2:6.6
	3 " 4	1:2.7:6.3	1:3.0:6.3	1:3.4:6.3	1:4.0:6.2	1:4.5:5.9
	6 " 7	1:2.1:5.1	1:2.4:5.2	1:2.7:5.2	1:3.1:6.1	1:3.5:4.9
	8 "10	1:1.4:3.8	1:1.6:3.9	1:1.8:3.9	1:2.1:3.9	1:2.4:3.8

Proportions for 2 000 Lb. per Sq. In. Concrete.

Proportions are expressed by volume as follows: Portland cement; fine aggregate; coarse aggregate.

Thus, 1:2.6:4.6 indicates 1 part by volume of Portland cement, 2.6 parts by volume of fine aggregate and 4.6 parts by volume of coarse aggregate.

Size of coarse aggregate	Slump, in inches	Size of Fine Aggregate				
		0 – No. 28	0 – No. 14	0 – No. 8	0 – No. 4	0 – ⅜ in.
None	½ to 1	1 : 2.2	1 : 2.6	1 : 3.0	1 : 3.5	1 : 4.1
	3 " 4	1 : 1.9	1 : 2.2	1 : 2.6	1 : 3.0	1 : 3.5
	6 " 7	**1 : 1.5**	**1 : 1.7**	**1 : 2.0**	**1 : 2.3**	**1 : 2.7**
	8 "10	**1 : 1.0**	**1 : 1.1**	**1 : 1.3**	1 : 1.6	1 : 1.8
No. 4 to ¾ in.	½ to 1	1 : 2.1 : 3.8	1 : 2.3 : 3.7	1 : 2.6 : 3.5	1 : 3.0 : 3.1	1 : 3.6 : 2.8
	3 " 4	1 : 1.7 : 3.3	1 : 1.9 : 3.2	1 : 2.2 : 3.1	1 : 2.6 : 2.8	1 : 3.0 : 2.4
	6 " 7	**1 : 1.3 : 2.7**	**1 : 1.4 : 2.6**	**1 : 1.7 : 2.5**	**1 : 1.9 : 2.3**	**1 : 2.3 : 2.1**
	8 "10	1 : 0.8 : 1.9	1 : 0.9 : 1.9	1 : 1.0 : 1.8	1 : 1.2 : 1.7	1 : 1.5 : 1.6
No. 4 to 1 in.	½ to 1	1 : 1.9 : 4.5	1 : 2.2 : 4.3	1 : 2.5 : 4.2	1 : 2.8 : 3.9	1 : 3.4 : 3.6
	3 " 4	1 : 1.6 : 3.9	1 : 1.8 : 3.8	1 : 2.1 : 3.7	1 : 2.4 : 3.5	1 : 2.8 : 3.2
	6 " 7	**1 : 1.2 : 3.1**	**1 : 1.3 : 3.1**	**1 : 1.5 : 3.0**	**1 : 1.8 : 2.9**	**1 : 2.1 : 2.7**
	8 "10	1 : 0.7 : 2.2	1 : 0.8 : 2.2	1 : 1.0 : 2.3	1 : 1.1 : 2.1	1 : 1.3 : 2.0
No. 4 to 1½ in.	½ to 1	1 : 1.9 : 5.0	1 : 2.1 : 4.9	1 : 2.4 : 4.9	1 : 2.7 : 4.6	1 : 3.2 : 4.4
	3 " 4	1 : 1.6 : 4.4	1 : 1.7 : 4.3	1 : 2.0 : 4.2	1 : 2.4 : 4.0	1 : 2.7 : 3.8
	6 " 7	**1 : 1.1 : 3.5**	**1 : 1.3 : 3.5**	**1 : 1.4 : 3.5**	**1 : 1.7 : 3.4**	**1 : 2.0 : 3.2**
	8 "10	1 : 0.7 : 2.5	1 : 0.8 : 2.5	1 : 0.9 : 2.5	1 : 1.0 : 2.4	1 : 1.2 : 2.3
No. 4 to 2 in.	½ to 1	1 : 1.7 : 5.8	1 : 1.9 : 5.7	1 : 2.1 : 5.8	1 : 2.4 : 5.6	1 : 2.8 : 5.5
	3 " 4	1 : 1.4 : 5.0	1 : 1.5 : 5.0	1 : 1.8 : 5.0	1 : 2.0 : 4.9	1 : 2.3 : 4.7
	6 " 7	**1 : 1.0 : 4.1**	**1 : 1.1 : 4.1**	**1 : 1.2 : 4.1**	**1 : 1.4 : 4.1**	**1 : 1.7 : 3.9**
	8 "10	1 : 0.6 : 2.9	1 : 0.7 : 2.9	1 : 0.7 : 3.0	1 : 0.8 : 2.9	1 : 1.0 : 2.9
⅜ to 1 in.	½ to 1	1 : 2.2 : 4.4	1 : 2.5 : 4.2	1 : 2.8 : 4.1	1 : 3.3 : 3.8	1 : 3.8 : 3.4
	3 " 4	1 : 1.9 : 3.8	1 : 2.1 : 3.7	1 : 2.4 : 3.6	1 : 2.8 : 3.4	1 : 3.2 : 3.1
	6 " 7	**1 : 1.4 : 3.1**	**1 : 1.5 : 3.0**	**1 : 1.8 : 3.0**	**1 : 2.1 : 2.8**	**1 : 2.4 : 2.5**
	8 "10	1 : 0.9 : 2.2	1 : 1.0 : 2.2	1 : 1.1 : 2.2	1 : 1.3 : 2.0	1 : 1.5 : 1.9
⅜ to 1½ in.	½ to 1	1 : 2.2 : 4.9	1 : 2.5 : 4.8	1 : 2.8 : 4.7	1 : 3.2 : 4.6	1 : 3.7 : 4.2
	3 " 4	1 : 1.9 : 4.3	1 : 2.1 : 4.2	1 : 2.4 : 4.1	1 : 2.7 : 4.0	1 : 3.1 : 3.7
	6 " 7	**1 : 1.4 : 3.5**	**1 : 1.5 : 3.4**	**1 : 1.7 : 3.4**	**1 : 2.0 : 3.3**	**1 : 2.3 : 3.1**
	8 "10	1 : 0.9 : 2.5	1 : 1.0 : 2.5	1 : 1.1 : 2.4	1 : 1.3 : 2.4	1 : 1.5 : 2.3
⅜ to 2 in.	½ to 1	1 : 2.1 : 5.6	1 : 2.3 : 5.5	1 : 2.6 : 5.5	1 : 3.0 : 5.4	1 : 3.5 : 5.1
	3 " 4	1 : 1.7 : 4.8	1 : 2.0 : 4.8	1 : 2.2 : 4.8	1 : 2.5 : 4.7	1 : 2.9 : 4.4
	6 " 7	**1 : 1.3 : 4.0**	**1 : 1.4 : 3.9**	**1 : 1.6 : 3.9**	**1 : 1.8 : 3.9**	**1 : 2.1 : 3.8**
	8 "10	1 : 0.8 : 2.9	1 : 0.9 : 2.9	1 : 1.0 : 2.9	1 : 1.2 : 2.9	1 : 1.3 : 2.8
¾ to 1½ in.	½ to 1	1 : 2.6 : 4.5	1 : 2.9 : 4.5	1 : 3.3 : 4.4	1 : 3.8 : 4.2	1 : 4.3 : 3.9
	3 " 4	1 : 2.2 : 3.9	1 : 2.5 : 3.9	1 : 2.8 : 3.8	1 : 3.2 : 3.6	1 : 3.6 : 3.3
	6 " 7	**1 : 1.6 : 3.2**	**1 : 1.8 : 3.2**	**1 : 2.1 : 3.1**	**1 : 2.4 : 3.0**	**1 : 2.7 : 2.8**
	8 "10	1 : 1.0 : 2.3	1 : 1.2 : 2.3	1 : 1.4 : 2.2	1 : 1.6 : 2.2	1 : 1.8 : 2.1
¾ to 2 in.	½ to 1	1 : 2.5 : 5.2	1 : 2.8 : 5.2	1 : 3.2 : 5.1	1 : 3.6 : 5.0	1 : 4.1 : 4.7
	3 " 4	1 : 2.1 : 4.5	1 : 2.4 : 4.5	1 : 2.7 : 4.4	1 : 3.1 : 4.3	1 : 3.5 : 4.0
	6 " 7	**1 : 1.6 : 3.7**	**1 : 1.8 : 3.7**	**1 : 2.0 : 3.7**	**1 : 2.3 : 3.6**	**1 : 2.6 : 3.5**
	8 "10	1 : 1.0 : 2.6	1 : 1.1 : 2.7	1 : 1.3 : 2.6	1 : 1.5 : 2.7	1 : 1.7 : 2.6
¾ to 3 in.	½ to 1	1 : 2.5 : 6.0	1 : 2.9 : 5.9	1 : 3.2 : 5.9	1 : 3.6 : 5.8	1 : 4.1 : 5.6
	3 " 4	1 : 2.1 : 5.1	1 : 2.4 : 5.2	1 : 2.7 : 5.2	1 : 3.1 : 5.1	1 : 3.5 : 4.9
	6 " 7	**1 : 1.5 : 4.1**	**1 : 1.7 : 4.2**	**1 : 2.0 : 4.2**	**1 : 2.3 : 4.2**	**1 : 2.5 : 4.0**
	8 "10	1 : 1.0 : 2.9	1 : 1.1 : 3.0	1 : 1.3 : 3.0	1 : 1.5 : 3.0	1 : 1.7 : 3.0

Proportions for 2 500 Lb. per Sq. In. Concrete.

Proportions are expressed by volume as follows: Portland cement; fine aggregate; coarse aggregate.

Thus, 1:2.6:4.6 indicates 1 part by volume of Portland cement, 2.6 parts by volume of fine aggregate and 4.6 parts by volume of coarse aggregate.

Size of coarse aggregate	Slump, in inches	Size of Fine Aggregate				
		0 – No. 28	0 – No. 14	0 – No. 8	0 – No. 4	0 – ⅜ in.
None	½ to 1	1 : 1.8	1 : 2.1	1 : 2.4	1 : 2.9	1 : 3.3
	3 " 4	1 : 1.5	1 : 1.8	1 : 2.1	1 : 2.4	1 : 2.8
	6 " 7	**1 : 1.1**	**1 : 1.3**	**1 : 1.6**	**1 : 1.8**	**1 : 2.1**
	8 "10	1 : 0.7	1 : 0.8	1 : 0.9	1 : 1.1	1 : 1.3
No. 4 to ¾ in.	½ to 1	1 : 1.6 : 3.2	1 : 1.8 : 3.1	1 : 2.1 : 3.0	1 : 2.4 : 2.7	1 : 2.9 : 2.4
	3 " 4	1 : 1.3 : 2.8	1 : 1.5 : 2.7	1 : 1.7 : 2.6	1 : 2.0 : 2.4	1 : 2.4 : 2.2
	6 " 7	**1 : 1.0 : 2.2**	**1 : 1.1 : 2.2**	**1 : 1.3 : 2.1**	**1 : 1.5 : 2.0**	**1 : 1.8 : 1.8**
	8 "10	1 : 0.5 : 1.4	1 : 0.6 : 1.4	1 : 0.7 : 1.4	1 : 0.8 : 1.4	1 : 1.0 : 1.3
No. 4 to 1 in.	½ to 1	1 : 1.5 : 3.7	1 : 1.7 : 3.7	1 : 2.0 : 3.5	1 : 2.2 : 3.4	1 : 2.7 : 3.1
	3 " 4	1 : 1.2 : 3.3	1 : 1.4 : 3.2	1 : 1.6 : 3.1	1 : 1.9 : 3.0	1 : 2.2 : 2.7
	6 " 7	**1 : 0.9 : 2.6**	**1 : 1.0 : 2.5**	**1 : 1.1 : 2.5**	**1 : 1.3 : 2.4**	**1 : 1.6 : 2.3**
	8 "10	1 : 0.5 : 1.7	1 : 0.6 : 1.7	1 : 0.6 : 1.7	1 : 0.7 : 1.6	1 : 0.9 : 1.5
No. 4 to 1½ in.	½ to 1	1 : 1.4 : 4.2	1 : 1.6 : 4.1	1 : 1.9 : 4.1	1 : 2.2 : 4.0	1 : 2.5 : 3.8
	3 " 4	1 : 1.2 : 3.7	1 : 1.3 : 3.6	1 : 1.5 : 3.6	1 : 1.8 : 3.5	1 : 2.1 : 3.3
	6 " 7	**1 : 0.9 : 2.9**	**1 : 0.9 : 2.8**	**1 : 1.1 : 2.8**	**1 : 1.3 : 2.8**	**1 : 1.5 : 2.6**
	8 "10	1 : 0.5 : 1.9	1 : 0.5 : 1.9	1 : 0.6 : 1.9	1 : 0.7 : 1.8	1 : 0.8 : 1.8
No. 4 to 2 in.	½ to 1	1 : 1.3 : 4.9	1 : 1.4 : 4.8	1 : 1.6 : 4.9	1 : 1.9 : 4.8	1 : 2.2 : 4.7
	3 " 4	1 : 1.1 : 4.3	1 : 1.2 : 4.2	1 : 1.3 : 4.3	1 : 1.6 : 4.2	1 : 1.8 : 4.1
	6 " 7	**1 : 0.7 : 3.3**	**1 : 0.8 : 3.3**	**1 : 0.9 : 3.4**	**1 : 1.1 : 3.3**	**1 : 1.2 : 3.3**
	8 "10	1 : 0.4 : 2.2	1 : 2.4 : 2.2	1 : 0.5 : 2.2	1 : 0.6 : 2.2	1 : 0.6 : 2.2
⅜ to 1 in.	½ to 1	1 : 1.8 : 3.7	1 : 2.0 : 3.6	1 : 2.3 : 3.5	1 : 2.6 : 3.3	1 : 3.0 : 2.9
	3 " 4	1 : 1.4 : 3.2	1 : 1.6 : 3.1	1 : 1.9 : 2.9	1 : 2.2 : 2.9	1 : 2.5 : 2.6
	6 " 7	**1 : 1.0 : 2.5**	**1 : 1.2 : 2.5**	**1 : 1.3 : 2.4**	**1 : 1.6 : 2.3**	**1 : 1.8 : 2.2**
	8 "10	1 : 0.6 : 1.6	1 : 0.7 : 1.6	1 : 0.8 : 1.6	1 : 0.9 : 1.6	1 : 1.0 : 1.5
⅜ to 1½ in.	½ to 1	1 : 1.7 : 4.1	1 : 1.9 : 4.1	1 : 2.2 : 4.0	1 : 2.5 : 3.9	1 : 2.9 : 3.6
	3 " 4	1 : 1.5 : 3.6	1 : 1.6 : 3.6	1 : 1.8 : 3.5	1 : 2.1 : 3.4	1 : 2 3 : 3.2
	6 " 7	**1 : 1.0 : 2.9**	**1 : 1.2 : 2.8**	**1 : 1.3 : 2.8**	**1 : 1.5 : 2.7**	**1 : 1.8 : 2.6**
	8 "10	1 : 0.6 : 1.9	1 : 0.6 : 1.9	1 : 0.8 : 1.8	1 : 0.9 : 1.8	1 : 1.0 : 1.8
⅜ to 2 in.	½ to 1	1 : 1.7 : 4.7	1 : 1.8 : 4.7	1 : 2.1 : 4.7	1 : 2.4 : 4.6	1 : 2.7 : 4.4
	3 " 4	1 : 1.4 : 4.1	1 : 1.5 : 4.1	1 : 1.7 : 4.1	1 : 2.0 : 4.0	1 : 2.3 : 3.9
	6 " 7	**1 : 1.0 : 3.2**	**1 : 1.1 : 3.2**	**1 : 1.2 : 3.2**	**1 : 1.4 : 3.2**	**1 : 1.6 : 3.1**
	8 "10	1 : 0.5 : 2.1	1 : 0.6 : 2.1	1 : 0.7 : 2.2	1 : 0.8 : 2.2	1 : 0.9 : 2.1
¾ to 1½ in.	½ to 1	1 : 2.0 : 3.8	1 : 2.3 : 3.8	1 : 2.6 : 3.7	1 : 3.0 : 3.6	1 : 3.4 : 3.3
	3 " 4	1 : 1.7 : 3.3	1 : 2.0 : 3.3	1 : 2.2 : 3.2	1 : 2.5 : 3.2	1 : 2.9 : 2.9
	6 " 7	**1 : 1.2 : 2.6**	**1 : 1.4 : 2.6**	**1 : 1.6 : 2.6**	**1 : 1.9 : 2.5**	**1 : 2.1 : 2.3**
	8 "10	1 : 0.7 : 1.7	1 : 0.8 : 1.7	1 : 0.9 : 1.7	1 : 1.1 : 1.7	1 : 1.2 : 1.6
¾ to 2 in.	½ to 1	1 : 2.0 : 4.4	1 : 2.2 : 4.4	1 : 2.5 : 4.3	1 : 2.9 : 4.3	1 : 3.3 : 4.1
	3 " 4	1 : 1.7 : 3.8	1 : 1.9 : 3.8	1 : 2.1 : 3.8	1 : 2.5 : 3.7	1 : 2.8 : 3.6
	6 " 7	**1 : 1.2 : 3.0**	**1 : 1.4 : 3.0**	**1 : 1.5 : 3.0**	**1 : 1.8 : 3.0**	**1 : 2.0 : 2.8**
	8 "10	1 : 0.7 : 2.0	1 : 0.8 : 2.0	1 : 0.9 : 2.0	1 : 1.0 : 2.0	1 : 1.2 : 2.0
¾ to 3 in.	½ to 1	1 : 2.0 : 5.0	1 : 2.2 : 5.0	1 : 2.5 : 5.0	1 : 2.7 : 5.0	1 : 3.2 : 4.7
	3 " 4	1 : 1.7 : 4.3	1 : 1.9 : 4.3	1 : 2.1 : 4.3	1 : 2.4 : 4.3	1 : 2.7 : 4.1
	6 " 7	**1 : 1.2 : 3.3**	**1 : 1.4 : 3.4**	**1 : 1.5 : 3.4**	**1 : 1.8 : 3.4**	**1 : 2.0 : 3.3**
	8 "10	1 : 0.7 : 2.2	1 : 0 8 : 2.2	1 : 0.9 : 2.2	1 : 1.0 : 2.3	1 : 1.2 : 2.3

Proportions for 3 000 Lb. per Sq. In. Concrete.

Proportions are expressed by volume as follows: Portland cement; fine aggregate; coarse aggregate.

Thus, 1:2.6:4.6 indicates 1 part by volume of Portland cement, 2.6 parts by volume of fine aggregate and 4.6 parts by volume of coarse aggregate.

Size of coarse aggregate	Slump, in inches	Size of Fine Aggregate				
		0 – No. 28	0 – No. 14	0 – No. 8	0 – No. 4	0 – ⅜ in.
None	½ to 1	1 : 1.5	1 : 1.7	1 : 2.0	1 : 2.3	1 : 2.7
	3 " 4	1 : 1.2	1 : 1.4	1 : 1.7	1 : 1.9	1 : 2.3
	6 " 7	1 : 0.9	1 : 1.0	1 : 1.2	1 : 1.4	1 : 1.6
	8 "10	1 : 0.5	1 : 0.6	1 : 0.7	1 : 0.8	1 : 0.9
No. 4 to ¾ in.	½ to 1	1 : 1.3 : 2.7	1 : 1.5 : 2.6	1 : 1.7 : 2.5	1 : 1.9 : 2.4	1 : 2.3 : 2.1
	3 " 4	1 : 1.0 : 2.3	1 : 1.2 : 2.2	1 : 1.4 : 2.2	1 : 1.6 : 2.0	1 : 1.9 : 1.8
	6 " 7	1 : 0.7 : 1.7	1 : 0.8 : 1.7	1 : 0.9 : 1.7	1 : 1.1 : 1.6	1 : 1.3 : 1.4
	8 "10	1 : 0.3 : 1.0	1 : 0.4 : 1.0	1 : 0.5 : 1.0	1 : 0.5 : 1.0	1 : 0.6 : 0.9
No. 4 to 1 in.	½ to 1	1 : 1.2 : 3.1	1 : 1.3 : 3.1	1 : 1.5 : 3.0	1 : 1.8 : 2.9	1 : 2.1 : 2.7
	3 " 4	1 : 0.9 : 2.7	1 : 1.1 : 2.6	1 : 1.2 : 2.6	1 : 1.4 : 2.5	1 : 1.7 : 2.3
	6 " 7	1 : 0.6 : 2.0	1 : 0.7 : 2.0	1 : 0.8 : 2.0	1 : 0.9 : 1.9	1 : 1.1 : 1.8
	8 "10	1 : 0.3 : 1.2	1 : 0.3 : 1.2	1 : 0.4 : 1.2	1 : 0.5 : 1.2	1 : 0.6 : 1.2
No. 4 to 1½ in.	½ to 1	1 : 1.1 : 3.6	1 : 1.2 : 3.5	1 : 1.5 : 3.5	1 : 1.7 : 3.4	1 : 2.0 : 3.2
	3 " 4	1 : 0.9 : 3.0	1 : 1.0 : 2.9	1 : 1.2 : 2.9	1 : 1.4 : 2.9	1 : 1.6 : 2.7
	6 " 7	1 : 0.6 : 2.2	1 : 0.7 : 2.2	1 : 0.8 : 2.2	1 : 0.9 : 2.2	1 : 1.1 : 2.1
	8 "10	1 : 0.3 : 1.4	1 : 0.3 : 1.3	1 : 0.4 : 1.4	1 : 0.5 : 1.4	1 : 0.5 : 1.3
No. 4 to 2 in.	½ to 1	1 : 1.0 : 4.1	1 : 1.1 : 4.1	1 : 1.2 : 4.1	1 : 1.4 : 4.1	1 : 1.6 : 4.0
	3 " 4	1 : 0.8 : 3.4	1 : 0.9 : 3.4	1 : 1.0 : 3.5	1 : 1.1 : 3.4	1 : 1.3 : 3.4
	6 " 7	1 : 0.5 : 2.6	1 : 0.6 : 2.6	1 : 0.6 : 2.7	1 : 0.7 : 2.6	1 : 0.9 : 2.6
	8 "10	1 : 0.2 : 1.6	1 : 0.3 : 1.6	1 : 0.3 : 1.7	1 : 0.4 : 1.7	1 : 0.4 : 1.7
⅜ to 1 in.	½ to 1	1 : 1.4 : 3.1	1 : 1.5 : 3.0	1 : 1.8 : 2.9	1 : 2.1 : 2.8	1 : 2.4 : 2.6
	3 " 4	1 : 1.1 : 2.6	1 : 1.3 : 2.6	1 : 1.5 : 2.5	1 : 1.7 : 2.4	1 : 2.0 : 2.2
	6 " 7	1 : 0.8 : 2.0	1 : 0.8 : 2.0	1 : 1.0 : 1.9	1 : 1.1 : 1.9	1 : 1.3 : 1.8
	8 "10	1 : 0.4 : 1.2	1 : 0.4 : 1.2	1 : 0.5 : 1.2	1 : 0.6 : 1.2	1 : 0.7 : 1.1
⅜ to 1½ in.	½ to 1	1 : 1.4 : 3.5	1 : 1.5 : 3.4	1 : 1.7 : 3.4	1 : 2.0 : 3.3	1 : 2.3 : 3.1
	3 " 4	1 : 1.1 : 3.0	1 : 1.2 : 2.9	1 : 1.4 : 2.9	1 : 1.6 : 2.8	1 : 1.9 : 2.6
	6 " 7	1 : 0.6 : 2.2	1 : 0.8 : 2.2	1 : 1.0 : 2.2	1 : 1.1 : 2.1	1 : 1.3 : 2.0
	8 "10	1 : 0.4 : 1.4	1 : 0.4 : 1.4	1 : 0.5 : 1.4	1 : 0.6 : 1.3	1 : 0.7 : 1.3
⅜ to 2 in.	½ to 1	1 : 1.3 : 4.0	1 : 1.4 : 4.0	1 : 1.6 : 4.0	1 : 1.9 : 3.9	1 : 2.1 : 3.8
	3 " 4	1 : 1.0 : 3.4	1 : 1.2 : 3.4	1 : 1.3 : 3.3	1 : 1.5 : 3.3	1 : 1.7 : 3.2
	6 " 7	1 : 0.7 : 2.6	1 : 0.8 : 2.5	1 : 0.9 : 2.6	1 : 1.0 : 2.6	1 : 1.1 : 2.5
	8 "10	1 : 0.4 : 1.6	1 : 0.4 : 1.6	1 : 0.5 : 1.6	1 : 0.5 : 1.6	1 : 0.6 : 1.6
¾ to 1½ in.	½ to 1	1 : 1.6 : 3.2	1 : 1.8 : 3.2	1 : 2.1 : 3.2	1 : 2.4 : 3.1	1 : 2.7 : 2.9
	3 " 4	1 : 1.3 : 2.7	1 : 1.5 : 2.7	1 : 1.7 : 2.7	1 : 2.0 : 2.6	1 : 2.3 : 2.5
	6 " 7	1 : 0.9 : 2.0	1 : 1.0 : 2.1	1 : 1.2 : 2.0	1 : 1.4 : 2.0	1 : 1.5 : 1.8
	8 "10	1 : 0.5 : 1.2	1 : 0.5 : 1.3	1 : 0.6 : 1.3	1 : 0.7 : 1.3	1 : 0.8 : 1.2
¾ to 2 in.	½ to 1	1 : 1.6 : 3.7	1 : 1.8 : 3.7	1 : 2.0 : 3.7	1 : 2.4 : 3.6	1 : 2.6 : 3.5
	3 " 4	1 : 1.3 : 3.1	1 : 1.5 : 3.1	1 : 1.6 : 3.1	1 : 1.9 : 3.1	1 : 2.2 : 3.0
	6 " 7	1 : 0.9 : 2.4	1 : 1.1 : 2.4	1 : 1.1 : 2.4	1 : 1.3 : 2.4	1 : 1.5 : 2.3
	8 "10	1 : 0.5 : 1.5	1 : 0.5 : 1.5	1 : 0.6 : 1.5	1 : 0.7 : 1.5	1 : 0.8 : 1.5
¾ to 3 in.	½ to 1	1 : 1.6 : 4.2	1 : 1.8 : 4.2	1 : 2.0 : 4.2	1 : 2.3 : 4.1	1 : 2.6 : 4.0
	3 " 4	1 : 1.3 : 3.5	1 : 1.5 : 3.6	1 : 1.6 : 3.6	1 : 1.9 : 3.6	1 : 2.1 : 3.5
	6 " 7	1 : 0.9 : 2.6	1 : 1.0 : 2.6	1 : 1.1 : 2.6	1 : 1.3 : 2.6	1 : 1.4 : 2.6
	8 "10	1 : 0.5 : 1.6	1 : 0.5 : 1.6	1 : 0.6 : 1.7	1 : 0.7 : 1.7	1 : 0.8 : 1.7

APPENDIX XVII

EFFECT OF OILS AND MISCELLANEOUS LIQUIDS ON CONCRETE AND METHOD OF PROTECTIVE TREATMENT WHERE REQUIRED.

Liquid.	Effect on untreated concrete.	Surface treatment.
Mineral Oils.*		
30 Baume or heavier....	Good concrete unaffected. Very slight surface penetration.	None. Good concrete, well spaded, or cement mortar finish sufficient.
Fuel oils above 30° Baume. Distillates, gas and lubricating oils†..	Good concrete unaffected. More penetration than for heavy oils.	Coatings of the magnesium fluo-silicate class, glues, or varnishes required for storage tanks.
Kerosene, gasoline, benzine.....	Good concrete unaffected. Considerable penetration,	Gasoline-proof coatings producing glazed surface or treatment with iron compounds.
Animal Oils (Solid Fats).‡		
Lard and lard oil.....	May attack concrete slowly, particularly if in melted condition.	Various proprietary compounds recommended by manufacturers of technical paints.
Goose fat, beef marrow, beef and mutton tallow, and tallow oil....	No definite information. Probably similar to lard oil.	Probably similar to that for lard oil.
Animal Oils (Liquid Fats).		
Marine:		
Menhaden oil.....	No effect on good concrete.	Dr. Cushman's tests indicate various coatings no better than plain concrete.
Cod liver oil.....	More or less disintegration, depending on quality of concrete.	Various proprietary compounds recommended by manufacturers of technical paints.
Shark liver oil.....	"	"
Seal and whale oil.....	"	"
Terrestrial:		
Sheep's foot.....	"	"
Horse's foot.....	"	"
Neat's foot§.....	No effect on good concrete.	No treatment required.
Vegetable Oils (Solid Fats).		
Cocoanut oil.....	Some action if stored in closed tank. Progressive disintegration if in contact with surfaces exposed to air.	Several proprietary compounds seem to have proved effective on floors. Sodium silicate or magnesium fluo-silicate treatment apparently sufficient for closed tanks.
Palm oil.....	No information.	No information.

* Signal oil, used by railroad companies, is a mixture of animal fat with mineral oil. Probably has about the same effect on concrete as lard oils.

† Some lubricating oils are mixtures of mineral and animal oils.

‡ U. S. Bureau of Standards tests with concrete tanks show slight roughening of surface at end of 12 months and considerable deposit on surface through saponification.

§ U. S. Bureau of Standards noted slight deposit due to saponification at end of 12 months.

‖ U. S. Bureau of Standards tests with concrete tanks show considerable softening and roughening of surface at end of 12 months.

APPENDIX XVII.—(*Continued.*)

Liquid.	Effect on untreated concrete.	Surface treatment.
Vegetable Oils.		
Drying:		
Hemp oil, Poppyseed oil, Tobacco oil	No information.	No information.
Linseed oil*, Rosin oil, Turpentine	No effect on good concrete. Considerable penetration of turpentine.	Cushman's tests indicate various coatings no better than plain concrete for linseed and rosin oils.
Semi-Drying:		
Cottonseed oil	No action if stored in closed tank of good concrete.	Same as for cocoanut oil.
Rape seed oil, Castor oil, Mustard oil	Progressive disintegration if in contact with surfaces exposed to air.	
Non-Drying:		
Olive oil	Probably some action.	Cushman's tests show proprietary coatings of varnish type effective.
Butter oil	.....	.....
Miscellaneous Liquids.		
Tanning liquors	Acid liquors show considerable effect. Other tanning extracts have no action.	Bituminous acid-proof paints effective for tanks holding acid tanning solutions. Good concrete with or without mortar finish sufficient for other tanning liquors.
Sulphite liquor	Attacks untreated concrete tanks.	Cushman's tests indicate bituminous acid-proof paints effective. Paraffin coating fair.
Cider vinegar	Acetic acid attacks concrete.	Cushman's tests indicate bituminous acid-proof paint effective. Paraffin coatings applied hot also useful for tanks.
Sauer-kraut brine	No action on good concrete.	Cushman's tests show special treatments no better than untreated concrete.
Whey	More or less action depending on quality of concrete.	Cushman's tests show proprietary coating of varnish type effective. Sodium silicate treatment used on storage tanks.
Buttermilk	No action on good concrete.	Untreated tanks used successfully to store buttermilk.
Molasses	No action in closed concrete.	Good concrete well spaded or finished with cement mortar sufficient. Annapolis mixture sometimes used.
Sulphuric acid solutions	Progressive disintegration, particularly where concrete is subject to abrasion.	Bituminous acid-proof or mastic coating effective.

* U. S. Bureau of Standards noted at end of 12 months considerable deposit on surfaces of concrete tanks containing both boiled and raw linseed oil, due to saponification, but concrete showed no deterioration.

APPENDIX XVIII

SUPERVISION AND INSPECTION OF CONCRETE CONSTRUCTION

1.—All work shall be under the supervision of the Engineer, under whose direction the work is to be conducted in accordance with the provisions of the specifications; the Engineer shall interpret the meaning and intent of the drawings and specifications, and pass on the material and workmanship, and his acceptance shall be a condition precedent to payment. In case of disputes, or of unforeseen conditions the decision of the Engineer as to quantities, quality, and acceptability of the work shall be binding.

2.—Line and grade will be given by the Engineer as provided under the contract. The Contractor shall inform the Engineer a reasonable time in advance so that lines and grades may be conveniently and accurately placed. Such points or marks as may be given by the Engineer shall be carefully preserved by the Contractor.

3.—The Inspector as a representative of the Engineer shall be in immediate charge of the inspection of material and of placing material in the finished work. The Contractor shall at all times give to the Owner, the Engineer, or Inspector, entrance to the work and to the place of manufacture of any materials entering into the work, and facilities for inspecting the work or materials, both at completion and in process of manufacture and construction. The manufacturer shall furnish the Engineer such chemical and physical records of the materials as may be required under the contract and any other records pertaining to the quality of the material that the manufacturer may possess shall be available for the information of the Engineer or Inspector.

4.—The Inspector shall be present before starting any unit of field work and during the process of construction of any unit of the structure, to see that the work is done according to the provisions of the specifications, as interpreted by the Engineer.

5.—The tests required under these specifications will be made by the Engineer and unless otherwise specifically provided for will be carried out at the expense of the Owner. The Contractor shall afford every opportunity for the proper conduct of the tests and he shall further provide such facilities for obtaining, handling, storing, and testing specimens and samples as the Engineer may require. For such work the Contractor shall receive compensation for actual expense for labor and materials.

6.—The inspection of the work shall not relieve the Contractor of his obligations under the contract. Defective work shall be made good and unsuitable materials rejected, even though such work and materials have previously been accepted.

THE INTERCONNECTED POWER SYSTEMS OF THE SOUTHEAST*

By Charles G. Adsit,† M. Am. Soc. C. E.

Synopsis

Many articles have appeared in the press and in scientific magazines during recent years on the super-power systems of the country, and much attention has been directed to what has been called the super-power system of the Southeastern States, as illustrating one of the best developed power systems in the United States. Some of these articles have discussed at great length the question of voltage control, switching troubles, and other operating difficulties to be encountered and solved. Others have contained actual set-ups for conditions five to ten years hence, with maps of new lines which should be built, and have included tables showing the vast saving which will annually result in the operation of such a system, such saving being the direct result of improved load factor, concentration of generating capacity in large plants at favorable locations, and utilization to the maximum possible extent of all available water power, even in times of high-water or flood conditions. The saving, expressed in dollars, under some of these plans is so great that there is a marked tendency to become over-enthusiastic as to the immediate need and possibility of the proposed system, and to forget the physical and financial problems involved in making predictions or drawing conclusions.

It should be remembered that the growth of a generating and transmission system in any section of the country is distinctly evolutionary in character, and that the interconnected transmission systems of the five States of the Southeast, which to-day present the nearest approach to what many writers mis-name a "super-power system" (with, perhaps, the one exception of the systems of California), were not built as a super-power system, but as an interconnection of various high-tension systems already in existence, so that the maximum interchange of power could take place between these systems as a whole, operated from a purely ecomonical standpoint. It exists to-day as a group of high-voltage transmission lines and generating plants which were begun as small units and were rapidly expanded until the distributing lines were nearly touching each other at various points, and the actual interconnection involved the construction of connecting lines of relatively short length. It is thus apparent that the interconnection of these various systems was not the prime consideration in their development, but a logical and purely secondary step. In some cases, the actual connection, rather than calling for additional expenditures,

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† Vice-Pres. and Executive Engr., Georgia Ry. & Power Co., Atlanta, Ga.

had to be regarded as a credit item, in that it obviated the necessity of duplicate or overlapping distributing circuits. In only a few cases have the connecting lines been built for their actual value as an interchange power connection between the two companies, without any consideration of the economic features of the proposed extensions based on the new loads to be served thereby.

These statements have been made with no desire to throw cold water on the various so-called super-power systems which have been proposed for different sections of the country, nor any lack of appreciation of the service which is being rendered the power industry by the various studies which engineers are devoting to this subject, but simply to suggest that such development as does take place along these lines in the next decade will be a logical one in which each step taken will be justified by the economic considerations at the time rather than the economies which will prevail when the completed system has been perfected.

The Georgia Railway and Power Company's System described herein is not the largest in the interconnected transmission systems of the Southeast, but its growth and plan for future extensions are typical of the other companies connected therewith.

In 1902 and 1903, the first water-power plant in this section was completed at Morgan Falls, on the Chattahoochee River, 16 miles from Atlanta, Ga., and a 22 000-volt line was built from this point to Atlanta by a company known as the Atlanta Water Power and Electric Company. This plant consists of a 48-ft., solid masonry dam about 1 000 ft. long, and a power-house which contained seven 2 500-h. p., horizontal type, water-wheels, operating under a head of 48 ft., direct connected to seven 1 500-kw., 3-phase 25-cycle, alternating current generators, having a total capacity of 10 500 kw., with the necessary transformers for stepping up the generating voltage of 2 300 to 22 000 volts. It was built for operation at 25 cycles, and had an annual rated output of approximately 30 000 000 kw-hr., provided the load could absorb this amount of power. It was regarded at that time as a very large hydro-electric development, and many famous engineers were consulted before its construction was undertaken, the names of whom appear on a bronze tablet in the plant. This plant is now being re-built as a 60-cycle plant, with a total installation of 15 000 kw. in generating equipment and 22 500 h. p. in water-wheels, by rewinding the generators and replacing the old water-wheels with new units of more modern design. Mention of this plant is made as being one of the first important water-power plants built in the South, and also in order to show the difference in output of a plant operated on a separately connected load, with the same plant connected to a larger system, such as that under consideration. The Morgan Falls plant was built with an expected output of 30 000 000 kw-hr., but after changing it to 60-cycle, and connecting it to this large power system, the annual output has become more than 60 000 000 kw-hr.

From 1902 to 1912, the whole Southeast, including the City of Atlanta, enjoyed a rapid growth, and the distributing lines around the city were extended from time to time to supply the increased demands of the industrial

and lighting loads, but the real impetus in the power development of the State came in the few years prior to 1912 with the organization of the Georgia Power Company, when it acquired the necessary lands and water rights and proceeded to develop the power in the Tallulah River, at Tallulah Falls, Ga. The proposed plant was to operate under a head of 608 ft. and have an immediate installation of 60 000 kw., with space provided for installation of an additional 12 000-kw. unit at a later date. The development attracted much attention on account of the high head involved, which at that time was the highest head utilized with any plant east of the Rocky Mountains, and on account of the local opposition which was encountered in prosecuting the work. For many years prior to this date Tallulah Falls had been regarded as one of the wonder spots of Nature, where the continual roar of the water passing over the Falls, the gorgeous mountain scenery of the section, and the delightful summer climate attracted many visitors each summer, and the idea of taking the water from the river and passing it silently through a large tunnel, penstocks, and water-wheels, to render a purely commercial service, rather than allow it to continue its natural noisy course from rock to rock over the precipitous falls at the head of the gorge, appeared to many to be little short of desecration, and some people, including the State authorities, spent large sums of money in attempting to prevent the development. The reader may laugh at this, forgetting that it was necessary to mount cannon on some of the first railroad trains to protect the trainmen from injury at the hands of people in the country through which it passed, and that practically every forward step in the world's progress has been bitterly opposed when first suggested.

It should be added at this point, however, that such opinions are not now generally held, and that even the people who most bitterly opposed the Tallulah project on the grounds that it would destroy Nature's handiwork are now convinced that the water of the Tallulah River is performing a greater service to mankind than before the development, as the power generated by this water is transmitted all over the State to help in lighting the homes of thousands of people, whereas before it could only be viewed or heard by a few who were fortunate enough to spend a holiday season at Tallulah Falls. Furthermore, it is universally agreed now that the mountain lakes which have been formed by the various dams above the Falls are equally as magnificent and have meant more to the immediate development of the surrounding country than the original Falls themselves.

The total generating capacity of the Georgia Railway and Power Company on January 1, 1914, was as follows:

	Kilowatts.
Tallulah Falls Plant (Hydro)	60 000
Morgan Falls Plant (Hydro)	10 500
Dunlap Plant (Hydro)	2 200
Butler Street Steam Plant, Atlanta	10 000
Davis Street Steam Plant, Atlanta	10 000
Total	92 700

The transmission system of the Company then consisted of two 110 000-volt circuits strung along a steel tower line from Tallulah Falls Station to Atlanta, 90 miles, a 50 000-volt, single circuit line on steel towers from the Dunlap plant near Gainesville, Ga., to Atlanta, about 50 miles distant, two 22 000-volt circuits on wood pole lines from the Morgan Falls plant to Atlanta, a distance of 16 miles. There was also one 110 000-volt circuit, 71 miles long, supported by steel towers between Atlanta and Lindale, Ga., for supplying the Lindale Station and intermediate points along this line, and one 110 000-volt circuit from Atlanta to Newnan, Ga., also supported by steel towers, a distance of 40 miles, these steel tower lines having been constructed in 1912 and 1913, as a part of the Tallulah Development. Transformer stations, of the most modern design, with capacities of 3 000 to 30 000 kw., were also erected to step down the voltage from 110 000 to 22 000 or 11 000 volts at Lindale, Marietta, Cartersville, Newnan, Gainesville, and Atlanta, Ga., for supplying the low-voltage distribution circuits to be extended from these points as the demand for power materialized.

The anticipated output of the Tallulah Falls plant at the time it was constructed was approximately 125 000 000 kw-hr. per annum, but the connected load on the Company's system at that time could not absorb this power, so a contract was negotiated with the Southern Power Company for a part of the output, and a steel tower line was built from Tallulah to Easley, S. C., supporting two 2/0 circuits, each good for a capacity of 20 000 kw., for delivering a part of the output from the new plant to the Southern Power System.

This 60-mile line, with a capacity of approximately 40 000 kw., was the first step in the interconnected system as it now stands, connecting the system of the Georgia Railway and Power Company with that of the Southern Power Company.

Following this initial high-voltage line construction, just described, arrangements were made for constructing a 1/0 copper circuit on wood poles from the Tennessee System at Cleveland, Tenn., into the northern part of the State of Georgia, to serve the territory around Dalton and Calhoun, and this line was finally extended to Lindale, to connect with the Georgia System at this point to serve as a tie line between the two systems as well as a distributing feeder for the intervening territory. It was designed for and is still operated at 60 000 volts, with a capacity of approximately 10 000 kw. The usefulness of this line was rapidly outgrown. A new 4/0, 110 000-volt, single circuit line was started in 1923 to connect Lindale with the Tennessee Power System at Chattanooga, Tenn., this line to have a capacity of 25 000 to 30 000 kw., in addition to the 60 000-volt line.

The Central Georgia Power Company, operating in the central part of the State around Macon, Ga., also soon afterward extended a 66 000-volt double circuit on steel towers from its new water-power plant at Lloyd Shoals, through Griffin, Ga., to the outskirts of Atlanta, where connection is made with the Georgia Railway and Power Company's System at a station known as the South Atlanta Station.

The Columbus Power Company, some years later, extended its transmission system from its water-power development at Goat Rock, near Columbus, Ga.,

to Newnan, to supply cotton-mill loads in the Newnan territory, and a short line about three miles long was next built between the sub-station of this and that of the Georgia Railway and Power Company at Newnan, for the purpose of tying these two generating systems together, and permitting the interchange of power between them. A daily interchange has taken place between the Columbus Power Company and the Georgia Railway and Power Company since this interconnection was made in 1914. This has resulted in an immense saving of power to the State, the Georgia Company taking all the excess hydro-power during nights, Sundays, and holidays, that the Columbus Power Company has had available, and returning to the Columbus Power Company during its heavy load hours in the day.

In 1920, plans which had been under way for some time were finally consummated, and construction was begun on a 110 000-volt, single circuit, tie line between the Gadsden Steam Station of the Alabama Power Company's System and the Lindale Station of the Georgia Railway and Power Company, which was completed and put under voltage during 1921.

On the completion of the Alabama tie line referred to, in 1921, the Georgia Railway and Power Company's transmission system became the central unit of a group of seven large power-generating and transmission companies, connecting on the north with the Tennessee Power Company, on the east with the Southern Power Company and the Carolina Light and Power Company, on the south with the Central Georgia Transmission Company and the Columbus Power Company, and on the west with the Alabama Power Company, by means of which the power generated on any of these systems can be delivered to the distribution lines of any of the other six systems.

The Georgia Company still operates the various small hydro-plants acquired and the steam plants at Atlanta, but its principal source of energy is the water power of the Tallulah and Tugalo Rivers, which will next be discussed in some detail.

The Tallulah and Chattooga Rivers unite to form the Tugalo, as shown on Fig. 1, and the latter joins the Seneca to form the Savannah, which forms the boundary line between Georgia and South Carolina. These rivers are fed by the many small streams draining the eastern slope of the Blue Ridge Mountains, where the altitude varies from 750 to 3 000 ft. above sea level. The territory surrounding these head-waters enjoys an annual precipitation on the average of approximately 70 in., a figure exceeded by few sections of the United States. It is mountainous in character, and densely wooded, with an annual run-off equal to approximately 60% of the rainfall. The fall in the streams is so great as to make them unsuited for any purpose except power development, and the region is so sparsely settled that little difficulties have been encountered in acquiring the land necessary for such development.

The original Tallulah Development previously mentioned consists of a cyclopean masonry dam, 116 ft. high and approximately 400 ft. long at the crest, built across the Tallulah River immediately above the well-known Tallulah Falls (Fig. 2). It serves to divert the water of the Tallulah River to an intake structure, built on the right bank of the stream a short distance up stream from the dam. The intake structure is a heavily reinforced, self-contained concrete

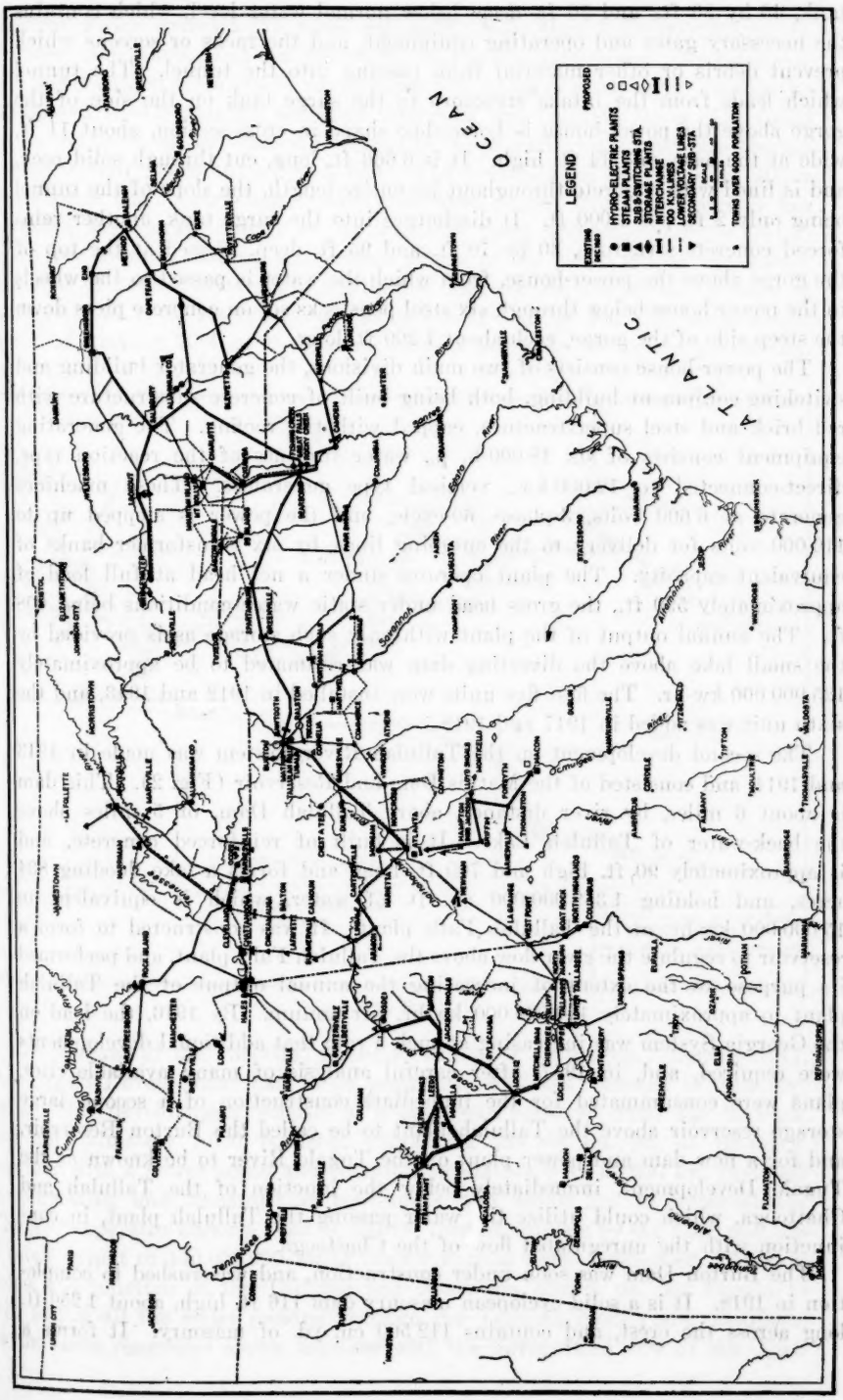
LEGEND
EXISTING
PROPOSED
HYDRO-ELECTRIC PLANTS
STEAM PLANTS
SUB & SWITCHING STA.
STORAGE PLANTS
INTERCHANGE
100 KV LINES
SECONDARY SUB-STA
TOWNS OVER 5000 POPULATION
ATLANTIC
OCEAN

FIG. 1.

tank, 40 by 80 ft., and 50 ft. deep, below normal water level, which contains the necessary gates and operating equipment, and the racks or screens which prevent débris or other material from passing into the tunnel. The tunnel which leads from the intake structure to the surge tank on the side of the gorge above the power-house is horse-shoe shape in cross-section, about 11 ft. wide at the base, and 14 ft. high. It is 6 666 ft. long, cut through solid rock, and is lined with concrete throughout its entire length, the slope of the tunnel being only 2 ft. per 1 000 ft. It discharges into the surge tank, another reinforced concrete structure, 30 by 70 ft. and 95 ft. deep, located at the top of the gorge above the power-house, from which the water is passed to the wheels in the power-house below through six steel penstocks set on concrete piers down the steep side of the gorge, each about 1 200 ft. long.

The power-house consists of two main divisions, the generator building and switching-equipment building, both being built of concrete substructure with red brick and steel superstructure, capped with tile roofing. The generating equipment consists of six 18 000-h. p., water turbines of the reaction type, direct-connected to 12 000-kw., vertical type generators. These machines generate at 6 600 volts, 3-phase, 60-cycle, and the power is stepped up to 110 000 volts for delivery to the outgoing lines, by six transformer banks of equivalent capacity. The plant operates under a net head at full load of approximately 590 ft., the gross head under static water conditions being 608 ft. The annual output of the plant with only such storage as is provided by the small lake above the diverting dam was estimated to be approximately 125 000 000 kw-hr. The first five units were installed in 1912 and 1913, and the sixth unit was added in 1917 and 1918.

The second development on the Tallulah River System was made in 1913 and 1914, and consisted of the Mathis Dam and Reservoir (Fig. 2). This dam is about 6 miles, by river distance, above Tallulah Dam, or 5 miles above the back-water of Tallulah Lake. It is built of reinforced concrete, and is approximately 90 ft. high and 700 ft. long, and forms a lake flooding 834 acres, and holding 1 369 000 000 cu. ft. of water, which is equivalent to 15 000 000 kw-hr. at the Tallulah Falls plant. It was constructed to form a reservoir to regulate the river flow above the Tallulah Falls plant, and performed its purpose to the extent of increasing the annual output of the Tallulah plant to approximately 140 000 000 kw-hr. per annum. By 1916, the load on the Georgia System was increasing at such a rate that additional developments were required, and, in 1917, after careful analysis of many available sites, plans were consummated for the immediate construction of a second large storage reservoir above the Tallulah plant to be called the Burton Reservoir, and for a new dam and power plant on the Tugalo River to be known as the Tugalo Development, immediately below the junction of the Tallulah and Chattooga, which could utilize the water passing the Tallulah plant, in conjunction with the unregulated flow of the Chattooga.

The Burton Dam was soon under construction, and was rushed to completion in 1918. It is a solid cyclopean masonry dam 116 ft. high, about 1 250 ft. long across the crest, and contains 112 500 cu. yd. of masonry. It forms a

reservoir which covers 2 775 acres and holds more than 5 000 000 000 cu. ft. of water, equivalent to 55 000 000 kw-hr. at the Tallulah plant. Its construction was justified by a study of hydrographs which showed that, during the spring and winter months, the flood waters of the Tallulah River that could not be retained by the existing reservoirs, were sufficient to fill the Burton Reservoir four years out of each five. The operation of the Tallulah River plants since the construction of this dam have proved that this reservoir has increased the annual output of the Tallulah plant approximately 50 000 000 kw-hr. per annum, making the present rating of the plant 190 000 000 kw-hr. in a year of normal water flow, with more than 200 000 000 kw-hr. in years of heavy rainfall, these reservoirs regulating the flow of the Tallulah River to the extent of making 90% of the annual run-off available for power generation.

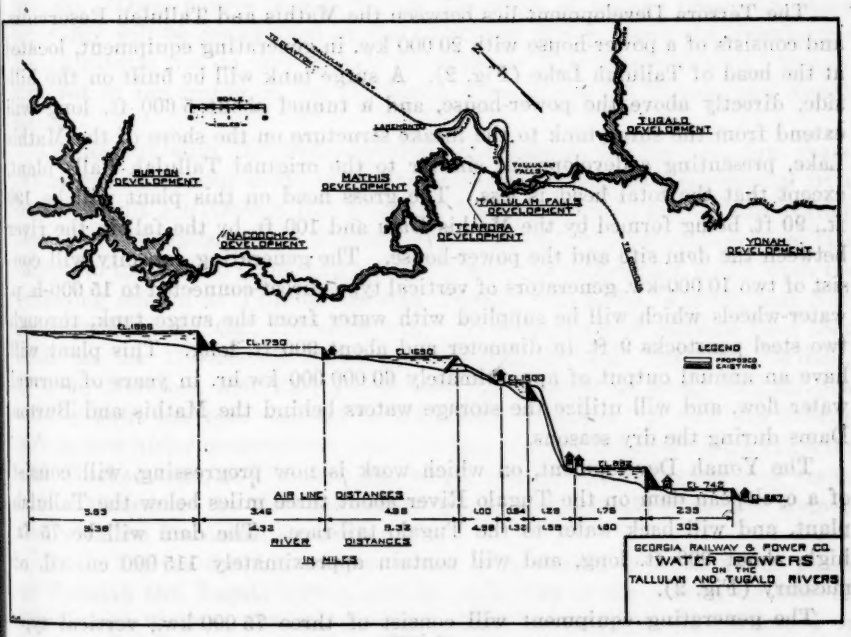

FIG. 2.

On account of the unsettled financial and labor conditions following 1917, the Tugalo Development had to be postponed, but was resumed in January, 1922. This development consists of a cyclopean masonry dam, 150 ft. high, and approximately 1 000 ft. long, containing 225 000 cu. yd. of masonry, together with a power-house and transformer station with 50 000 kw. installed in generating and transforming equipment, and 88 000 h.p. in water-wheels. The plant operates under a head of 150 ft., formed entirely by the dam, and has a rated output of 140 000 000 kw-hr. under normal water conditions. As it is below the junction of the Tallulah and Chattooga Rivers, on the Tugalo, it will utilize all the water passing through the Tallulah Falls plant, regulated by the three reservoirs above, together with the unregulated flow of the Chat-

tooga River, a stream approximately twice the size of the Tallulah. The lake formed by the dam itself has an area of 557 acres, and contains an available draw down of 500 000 000 cu. ft., which can be utilized for regulating the 24-hour flow of the Chattooga River. The plant contains four units of the vertical type, similar to the machines in the Tallulah plant, two of which were put in service in October, 1923, and the other two of which will be in service by April 1, 1924, or soon thereafter.

A study of load conditions early in 1923 disclosed the fact that the entire output of the Tugalo plant would be required by the load demands of the system immediately on completion, and, therefore, construction was begun in the early summer of 1923 on two more plants in the Tallulah and Tugalo chain, namely, the Terrora and Yonah Developments.

The Terrora Development lies between the Mathis and Tallulah Reservoirs, and consists of a power-house with 20 000 kw. in generating equipment, located at the head of Tallulah Lake (Fig. 2). A surge tank will be built on the hill-side, directly above the power-house, and a tunnel about 5 600 ft. long will extend from the surge tank to the intake structure on the shore of the Mathis Lake, presenting a development similar to the original Tallulah Falls plant, except that the total head is less. The gross head on this plant will be 190 ft., 90 ft. being formed by the Mathis Dam and 100 ft. by the fall in the river between the dam site and the power-house. The generating capacity will consist of two 10 000-kw. generators of vertical type, direct-connected to 15 000-h. p. water-wheels which will be supplied with water from the surge tank, through two steel penstocks 9 ft. in diameter and about 900 ft. long. This plant will have an annual output of approximately 60 000 000 kw-hr. in years of normal water flow, and will utilize the storage waters behind the Mathis and Burton Dams during the dry seasons.

The Yonah Development, on which work is now progressing, will consist of a cyclopean dam on the Tugalo River about three miles below the Tallulah plant, and will back water to the Tugalo tail-race. The dam will be 75 ft. high, about 900 ft. long, and will contain approximately 115 000 cu. yd. of masonry (Fig. 2).

The generating equipment will consist of three 75 000-kw., vertical type machines, direct-connected to 12 500-h.p. water-wheels, operating under a gross head of 75 ft. The plant will utilize all the water passing the Tugalo plant from the storage reservoirs on the Tallulah and the unregulated flow of the Chattooga, and is rated at about 70 000 000 kw-hr. per annum.

In addition to the developments described as now existing, or in course of construction, a smaller development will be started in the latter part of 1924, known as the Nacoochee plant, to utilize the 60-ft. fall on the Tallulah River between the Burton Dam and the back-water from the Mathis Dam, consisting of a 60-ft. dam and a power-house containing 5 000 kw. in a single unit. As soon as this plant is under way, a power-house with 6 000 kw. in generating equipment will be built below the Burton Dam, provision having already been made in the dam structure for the intake and penstock installation. These

plants are comparatively small, but will increase the total output of the system by approximately 50 000 000 kw-hr.

These plans provide for complete development of Tallulah and Tugalo Rivers throughout a river distance of 38 miles, where the total fall is 1 199 ft., practically every foot of which will be utilized in power production. The total installed capacity of the plants will be 178 000 kw., and the estimated annual output, during a year of normal water flow, will be 531 000 000 kw-hr., based on a net run-off of approximately 40 in. out of a total rainfall of 65 in. as the average over the entire water-shed. Attention is called to the fact that the water stored in the Burton and Mathis Reservoirs is equivalent to 125 000 000 kw-hr. at the water-power plants below the reservoirs, which is approximately 24% of the total annual output of the Tallulah and Tugalo System. These figures do not include the effect of storage reservoirs which some day will be constructed on the Chattooga River, above the Tugalo and Yonah plants; nor do they include the effect of reservoirs which will be constructed in the future on the Chattooga River above the Tugalo and Yonah plants.

Coincident with the hydro-electric developments of the past few years, extensions have been made to the transmission system by construction of a single circuit 110 000-volt line from the Terrora plant to a switching station near Tallulah Falls, with a double circuit line of the same voltage from this station to the Tugalo plant, and a single circuit, 110 000-volt, 4/0 line from the switching station at Tallulah through Toccoa, Commerce, Winder, and Norcross, to Marietta, Ga., 110 miles long. New transformer stations have been built on the 110 000-volt system at Cornelia, Toccoa, Commerce, Winder, Aragon, Norcross, and East Point, and the capacity of many stations previously built has been doubled or materially increased. A new 110 000-volt line is now under construction from Lindale through Summerville, LaFayette, and Chickamauga, to Chattanooga, which will be used to supply new transformer stations at the three first named points, and to establish another connection with the Tennessee System.

Table 1 gives the figures just quoted with respect to water-power plants on Tallulah and Tugalo Rivers, and the capacities of the various 110 000-volt transformer stations now in service or under construction, as illustrated on Fig. 1, which shows the Georgia System and its interconnections.

The total generating capacity of the Georgia Railway and Power Company's System, on the completion of the plants now under construction, and those immediately proposed, will be 217 400 kv-a., with an annual output of approximately 600 000 000 kw-hr., all of which will be in service within the next two or three years. To these figures should be added 189 000 kv-a. as the combined generating capacity of the Central Georgia and Columbus Power Company Systems, serving the central part of the State of Georgia, and directly connected with the Georgia System, giving 406 400 kv-a. as the total generating capacity which will be in service within the next few years on the interconnected transmission systems of the State. This does not include the small power developments or isolated steam plants which are not connected to the high-voltage transmission lines.

TABLE 1.—Data Relative to the Tallulah and Tugalo River Development.

Burton Development:	
Height of dam, in feet	116
Drainage area above dam, in square miles	136
Area of reservoir, in acres	2 775
Capacity of reservoir, in cubic feet	5 280 000 000
Capacity of reservoir, in equivalent kilowatt-hours:	
At Nacoochee Power-House	5 540 000
At Terrora Power-House	17 300 000
At Tallulah Falls Power-House	55 000 000
At Tugalo Power-House	13 850 000
At Yonah Power-House	8 600 000
Total	100 290 000
Generating capacity, in kilowatts	6 000
Water-wheel capacity, in horse-power	9 000
Head, in feet	84—116
Average annual output, in kilowatt-hours	32 200 000
Average annual output, in horse-power-hours	48 300 000
Nacoochee Development (Seed):	
Height of dam, in feet	60
Drainage area above dam, in square miles	144
Area of reservoir, in acres	560
Available capacity of reservoir, in cubic feet	350 000 000
Generating capacity, in kilowatts	5 000
Water-wheel capacity, in horse-power	7 500
Head, in feet	60
Average annual output, in kilowatt-hours	20 000 000
Average annual output, in horse-power-hours	30 000 000
Terrora Development (Mathis-Tallulah):	
Height of dam, in feet	90
Drainage area above dam, in square miles	153
Area of reservoir, in acres	884
Capacity of reservoir, in cubic feet	1 369 000 000
Capacity of reservoir, in equivalent kilowatt-hours:	
At Terrora Power-House	2 210 000
At Tallulah Falls Power-House	15 000 000
At Tugalo Power-House	3 940 000
At Yonah Power-House	2 450 000
Total	23 600 000
Generating capacity, in kilowatts	20 000
Water-wheel capacity, in horse-power	30 000
Head, in feet	190
Average annual output, in kilowatt-hours	63 800 000
Average annual output, in horse-power-hours	95 700 000
Tallulah Falls Development:	
Height of dam, in feet	116
Drainage area above dam, in square miles	190
Area of reservoir, in acres	63
Available capacity of reservoir, in cubic feet	63 000 000
Generating capacity, in kilowatts	72 000
Water-wheel capacity, in horse-power	108 000
Head, in feet	608
Average annual output, in kilowatt-hours	200 000 000
Average annual output, in horse-power-hours	300 000 000
Tugalo Development:	
Height of dam, in feet	140
Drainage area above dam, in square miles	481
Area of reservoir, in acres	557
Available capacity of reservoir, in cubic feet	500 000 000
Generating capacity, in kilowatts	59 000
Water-wheel capacity, in horse-power	88 000
Head, in feet	150
Average annual output, in kilowatt-hours	140 000 000
Average annual output, in horse-power-hours	210 000 000
Yonah Development (Ralston):	
Height of dam, in feet	75
Drainage area above dam, in square miles	490
Area of reservoir, in acres	425
Available capacity of reservoir, in cubic feet	350 000 000
Generating capacity, in kilowatts	25 000
Water-wheel capacity, in horse-power	37 500
Head, in feet	75
Average annual output, in kilowatt-hours	75 000 000
Average annual output, in horse-power-hours	112 500 000
Summary:	
Total storage capacity, in cubic feet	7 912 000 000
Total head, in feet	1 199
Total generating capacity, in kilowatts	178 000
Total water-wheel capacity, in horse-power	280 000
Total average annual output, in kilowatt-hours	531 000 000
Total average annual output, in horse-power-hours	796 500 000

As previously intimated, the Georgia System is the central system in the Southeastern network, being connected on the west with the Alabama System, on the north with the Tennessee System, and on the east with the Southern Power System, which, in turn, is connected with the Carolina Power and Light Company, operating in the territory northeast of the Southern Power System. The growth of the industry in the adjoining States has been equal to or greater than the development described.

In North Carolina and South Carolina, there will be a generating capacity amounting to 606 550 kv-a., on the completion of the plants listed in Table 2 under the Southern Power Company and the Carolina Power and Light Company. The Alabama System will have a total capacity of 571 800 kv-a. (not including the Muscle Shoals hydro-plant), and the Tennessee System, 196 405 kv-a., making the total combined generating capacity of the Southeastern System, on the completion of plants now under construction, or immediately proposed, 1 781 155 kv-a., with an estimated annual output not less than 5 000 000 000 kw-hr. The total output of the combined systems during 1923 is given in Table 3 as 2 979 543 600 kw-hr., which was 20% greater than the corresponding figure for the preceding year.

The territory within service distance of these various companies is approximately one-half the combined area of the States of North and South Carolina, Georgia, Alabama, and Tennessee, amounting to 120 000 sq. miles, with a total population of more than 6 000 000. It is thickly traversed with high-voltage transmission lines, thoroughly interconnected, and spotted every few miles with high-voltage transformer stations acting as supply points for the lower voltage distributing feeders radiating therefrom. Table 4 shows the approximate miles of high-voltage transmission circuits on the various systems, exclusive of all distributing lines operating at 44 000 volts, 38 000 volts, or less, from which it will be noted that the total actual line mileage of these high-voltage lines on the interconnected systems is 3 112, over which double circuits are strung in many cases, or provision is made for installation of a second circuit at a later date.

Thus far this paper has described the general interconnected power systems of the Southeast, and of the Georgia Railway and Power Company, in particular.

With regard to the operation of these interconnected power systems, little, if anything, of great interest can be said. In the abstract contemplation of super-power systems, much speculation and calculation is usually entered into in order to determine the operating characteristics and the physical operation, based on hypothetical systems and conditions. As a matter of fact, no unusual operating difficulties have been encountered. The system operates as a whole as simply as each system by itself, and the synchronizing and transference of power over these interconnections are carried out with the same regularity and with the same operating characteristics that is found in each of the systems individually. It is not even necessary to pay attention to the problem of voltage regulation, as each system, when interconnected, continues its regular operation without regard to the other systems. When two systems with somewhat varying voltages are connected, the voltage equalizes some-

TABLE 2.—Showing the Generating Capacity of the Various Companies Comprising the Southeastern Power System.

North Carolina and South Carolina.

Company.	Type of plant.	Existing capacity, in kilo-volt amperes.	Capacity under construction, in kilo-volt amperes.
Southern Power Company:			
Bridgewater	Hydro.	25 000	.....
Rhodiss	"	37 500	.....
Lookout	"	22 800	.....
Mountain Island	"	75 000	.....
Catawba	"	6 600	.....
99 Islands	"	22 500	.....
Fishing Creek	"	37 500	.....
Great Falls	"	30 000	.....
Rocky Creek	"	30 000	.....
Dearborn	"	56 250	.....
Wateree	"	70 000	.....
Total	.....	413 150	.....
Mt. Holly	Steam.	45 500	.....
Tiger	"	37 500	.....
Greenville	"	8 000	.....
Greensboro	"	8 000	.....
Eno	"	31 250	.....
Total	.....	130 250	.....
Total hydro and steam power	.....	543 350	.....
Carolina Power and Light Company:			
Buckhorn	Hydro.	2 900	.....
Blewetts	"	24 000	.....
Raleigh	Steam.	3 700	.....
Cape Fear	"	15 000	15 000
Florence	"	1 500	.....
Total steam power	.....	21 250	15 000
Total hydro power	.....	26 900	.....
Total hydro and steam power	.....	48 150	15 000

Note.—Some of the plants listed in Table 2 under Southern Power Company plants as existing are under construction. The table does not include all the proposed plants of the various companies, but only those on which construction has actually started.

TABLE 2.—(*Continued*).

TENNESSEE.

Company.	Type of plant.	Existing capacity, in kilovolt-amperes.	Capacity under construction, in kilovolt-amperes.
Tennessee Power Company:			
Hales Bar	Hydro	43 430	.....
Ocoee No. 1	"	21 000	.....
Ocoee No. 2	"	18 750	.....
Rock Island	"	9 375	15 000
Total hydro power	.....	92 555	15 000
Parksville	Steam.	13 000	.....
Nashville	"	29 600	.....
Knoxville	"	6 250	.....
Hales Bar	"	000	40 000
Total steam power	.....	48 850	40 000
Total hydro and steam power	.....	141 405	55 000

GEORGIA.

Georgia Railway and Power Company:			
Tallulah Falls	Hydro	72 000	.....
Tugalo	"	50 000	.....
Terrora	"	.....	20 000
Yonah	"	.....	25 000
Burton	"	.....	6 000
Nacoochee	"	.....	5 000
Morgan Falls	"	15 000	.....
Dunlap	"	2 200	.....
Chestatee	"	1 000	.....
Gregg Shoals	"	1 200	.....
Atlanta steam plants	.....	20 000	.....
Total hydro power	.....	141 400	56 000
Total steam power	.....	20 000	.....
Total hydro and steam power	.....	161 400	56 000
Central Georgia Power Company:			
Lloyd Shoals	Hydro	12 000	.....
Macon Plants	Steam	3 000	15 000
Total hydro and steam power	.....	15 000	15 000
Columbus Power Company:			
North Highlands	Hydro	6 900	53 100
Goat Rock	"	16 000	14 000
Bartletts Ferry	"	.....	60 000
Columbus Plant	Steam	9 000	.....
Total hydro power	.....	22 900	127 100
Total steam power	.....	9 000	.....
Total hydro and steam power	.....	31 900	127 100

TABLE 2.—(*Continued*).

ALABAMA.

Company.	Type of plant.	Existing capacity, in kilovolt-amperes.	Capacity under construction, in kilovolt-amperes.
Alabama Power Company:			
Lock 12	Hydro.	81 000	.....
Mitchell	"	60 000	.....
Jackson Shoals	"	2 000	.....
Upper Tallassee	"	.....	44 000
Lower Tallassee	"	.....	62 500
Cherokee	"	.....	150 000
Total hydro power	.....	143 000	256 500
Warrior River	Steam.	58 300	25 000
Gadsden	"	12 500	.....
Montgomery	"	6 000	.....
Sheffield	"	70 500	.....
Total steam power	.....	147 300	25 000
Total hydro and steam power	.....	290 300	281 500

SUMMARY.

Company.	EXISTING CAPACITY, OR CAPACITY UNDER CONSTRUCTION, IN KILOVOLT-AMPERES.		
	Hydro plants.	Steam plants.	Total.
Southern Power Company	413 150	130 250	543 400
Carolina Power and Light Company	26 900	36 250	63 150
Tennessee Power Company	107 555	88 850	196 405
Georgia Railway and Power Company	197 400	20 000	217 400
Central Georgia Power Company	12 000	18 000	30 000
Columbus Power Company	150 000	9 000	159 000
Alabama Power Company	399 500	172 300	517 800
Grand total, Southeastern System	1 406 505	474 650	1 781 155

where along the line of the interconnection, and the potential conditions on the separate systems are affected little, if any. There has been no trouble in staying in parallel, except during periods of light load, usually in the early morning hours, and if at that time any great amount of power is being transferred from one system to another, no trouble has been experienced.

Sections of the territory have been saved from serious interruptions of power service, several times during the period these companies have been so thoroughly interconnected. There have been times when many thousands of employees of cotton mills and other industries have been kept at work by a supply of power originating many hundreds of miles away. It is this insurance against shut-downs, when one company can call on the others for assistance, that the greatest benefit of the interconnection is derived, notwith-

standing there is a daily interchange of power throughout the territory which serves to conserve the water resources to a large extent. A word in passing should be added respecting the lease of the U. S. Government steam plant at Sheffield, Ala. Several of the power companies in the Southeast associated themselves in leasing this steam plant in order that it might be operated during the fall, or during dry weather, to supplement the water-power plants during the period immediately following the World War, when the hydraulic construc-

TABLE 3.—TOTAL POWER, IN KILOWATT-HOURS, ABSORBED BY SYSTEMS OF THE SOUTHEASTERN POWER COMPANIES.

Company.	1922.	1923.
Southern Power Company	910 488 432	1 001 026 319
Carolina Power and Light Company	160 584 432	198 452 248
Tennessee Power Company	351 436 289	460 097 505
Alabama Power Company	607 133 242	763 260 963
Georgia Railway and Power Company	286 049 535	347 818 196
Columbus Power Company	113 575 334	132 627 319
Central Georgia Power Company	65 021 460	76 261 050
Total	2 494 288 724	2 979 543 600

TABLE 4.—HIGH-VOLTAGE TRANSMISSION LINES OF THE SOUTHEASTERN POWER COMPANIES.*
(66 000 Volts; 100 000 Volts; and 110 000 Volts.)

Company.	Miles of line.
Southern Power Company	893.60
Carolina Power and Light Company	477.00
Tennessee Power Company	551.00
Alabama Power Company	534.23
Georgia Railway and Power Company	450.40
Columbus Power Company	71.10
Central Georgia Power Company	184.90
Grand total	3 112.23

* The distances represent length of lines, some carrying one or more circuits, and do not include any 44 000-volt, 38 000-volt, or lower voltage distributing circuits.

tion had not kept pace with the load. This plant has been operated each year by the Alabama Power Company, and power transmitted or relayed to the other companies in the Association up to the capacity of the lines connecting the Sheffield plant with the Alabama System. This is approximately 45 000 kw., and the steam plant has been operated at this load during the fall months for the past three years, with every indication that it will also be operated during the fall of 1924 to the same extent. Power in this steam plant has been relayed, not actually transmitted, from Sheffield, to Raleigh, N. C., a distance of nearly 900 miles. This relaying of power is one of the chief advantages of an interconnected system. Although the power from Sheffield was not actually used in Raleigh, the effect of the interconnected system

made it possible to use an equivalent amount of power in Raleigh, it having been relayed from one system to the other and used at the point of greatest advantage, involving the least distance of transmission. During this period, every system in the Southeast has been operated in parallel with every generating station on that system in service.

To an engineer familiar with the enormous investment required in the development of water-power construction of steam stations, and the erection of transmission lines operating at high voltage, the figures quoted give a graphic picture of the enormous development which has been going on during the past ten years in the industrial life of the Southeastern States, and that contemplated in the coming decade by those responsible for its power requirements. It would seem to the writer that the general super-power system must come after the various groups of States are interconnected the power companies of which have an immediate mutual interest, and when their development has grown to a point where the interconnection over wide territory will prove economical from the interchange and insurance of load standpoint, to the extent that will warrant the enormous expenditure involved in lines of large capacity and super-voltage.

THE SECONDARY EFFECT OF CERTAIN IMPORTANT RIVER BRIDGES ON LOCAL TRANSIT CONDITIONS

Discussion*

By John A. Miller, Jr., Assoc. M. Am. Soc. C. E.†

John A. Miller, Jr.,‡ Assoc. M. Am. Soc. C. E. (by letter).§—By presenting data concerning the vehicular traffic on important river bridges, Messrs. Fowler,‖ Steinman,‖ Lewis,‖ and Singstad¶ have added valuable information to the paper and broadened its scope. The discussion of tunnels *versus* bridges has also shed light on the question from a new direction. Both these important considerations were omitted from the original investigation because the study was made after the erection of a bridge in a particular location had been definitely decided upon, and because the problem was viewed only from the railway standpoint.

As Mr. Singstad has pointed out, the location of the Delaware River Bridge is probably not ideal as far as passenger traffic by rail is concerned; on the other hand to build a bridge where it would best serve the rail traffic would be impracticable on account of the resulting vehicular congestion at both ends. There is, therefore, much to recommend a tunnel. Even in that case, however, a problem would be presented concerning the best way to operate the facilities provided, and the conclusions already reached would be applicable.

The objections raised by Messrs. Weir** and Steinman to placing railway tracks on the bridge appear to exaggerate the importance of the automobile in the general transportation scheme. In the discussion it was said that "the capacity of the street is vastly increased by eliminating the trolley car." It was suggested that the capacity of a highway bridge would be increased if railway tracks were omitted from the design. The fact is, however, that the electric railway car, as far as efficient use of the street is concerned, surpasses any other means of transportation. Traffic counts recently made in six important cities show that the street cars are carrying upward of 75% of the total traffic, although they constitute only about 15% of the total number of vehicles. The figures are given in Table 1, from which it will be seen that the elimination of the electric railway car would increase the capacity of the street for automobiles, but would seriously reduce it for general transportation purposes. There seems to be no good reason why the automobile should

* Discussion of the paper by John A. Miller, Jr., Assoc. M. Am. Soc. C. E., continued from September, 1924, *Proceedings*.

† Author's closure.

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§ Received by the Secretary, August 15, 1924.

‖ *Proceedings*, Am. Soc. C. E., August, 1924, pp. 863, 869, 871.

¶ *Loc. cit.*, September, 1924, p. 1125.

** *Loc. cit.*, August, 1924, p. 865.

be especially favored in this respect, either in the street or on a bridge. The worth of a transportation facility on a bridge should be judged by the service it performs rather than by the tolls it provides.

TABLE 1.—TRAFFIC STATISTICS.

	Composition of Traffic.		Passengers Carried.	
	Railway cars. Percentage.	Other vehicles. Percentage.	By railway. Percentage.	By other vehicles. Percentage.
Chicago, Ill.	10	90	74	26
Baltimore, Md.	12	88	89	11
St. Louis, Mo.	10	90	75	25
Los Angeles, Calif.	25	75	92	8
Atlanta, Ga.	9	91	72	28
Montreal, Que., Canada	25	75	93	7

The statement that the original cost of a bridge such as that being built across the Delaware River, could be reduced as much as 50% by the elimination of railway tracks is true only in case the railway passengers elect to travel by an entirely different route. They could not be carried over the bridge by any other means that would produce as small a live load. The total weight per passenger carried, which is probably the best basis on which to compare the live load capacity required for various types of transportation units, is so nearly the same for a modern street car and an ordinary single-deck bus, that there is little to choose between them. Among the passenger automobiles, only a fully loaded Ford can compare with either of them in economy of weight per passenger. Typical weights are given in Table 2.

TABLE 2.—WEIGHTS OF TRANSPORTATION FACILITIES.

Class of facility.	Total weight, in pounds.	Passenger capacity.	Pounds per passenger.
Street Railway Cars:			
Birmingham Electric Company	33 800	110*	308
Connecticut Company	30 000	100*	300
Durham Public Service	27 000	90*	300
Eastern Texas Electric Company	26 000	85*	306
Northwestern Ohio Railway and Power Company	34 500	95*	363
Philadelphia Rapid Transit Company	34 000	100*	340
Motor Buses:			
Fageol	9 600	40†	240
Fifth Avenue, J	8 235	40†	206
Garford, 51-D	10 000	40†	250
Mack (low bus)	9 200	32†	288
White, 50-A	8 400	32†	262
Yellow, Z-29	10 000	40†	250
Passenger Automobiles:			
Ford Touring	1 650	5	330
Dodge "	2 546	5	509
Buick "	3 120	5	624
Nash "	3 400	5	608
Cadillac "	4 025	7	575
Packard "	4 746	7	678

* Estimate based on railway company's specifications.

† Estimate based on manufacturer's specifications.

Probably none of these facilities would be operated at its most efficient loading, except during the rush hours. Their relative merits, however, would not be altered on that account.

The slight advantage in weight possessed by the bus is more than counterbalanced by the road space occupied. This is a matter of paramount importance in the case of a bridge where the roadway is likely to be congested. The ordinary electric railway car occupies approximately 3.5 sq. ft. per passenger carried, a single-deck bus of average size about 5.0 sq. ft. per passenger, and a private automobile somewhat more than 20 sq. ft. per passenger. If every inch of space occupied by electric railway tracks on the East River bridges in New York, were devoted to passenger transportation by automotive vehicles, the passenger capacity would be woefully insufficient.

Moreover, experience has demonstrated times without number that traffic movement is accomplished most expeditiously with large units. In other words, 50 railway cars will cause far less confusion and congestion than 100 or more buses, regardless of the street space occupied, relative weights, or other details. Under these circumstances, the conclusion appears inevitable that if a bridge is to be used for mass transportation of passengers the means should be an electric railway.

ANALYTICAL SOLUTION OF MASONRY DOMES

Discussion*

By David C. Coyle, Esq.†

David C. Coyle,‡ Esq. (by letter).§—Mr. Whitney‖ calls attention to the fact that there may be bending stresses in the shell near the support and near the top, which would be dangerous for a thin shell. Where the dome has no plane of rupture, the yielding of the ring tends to straighten the profile just above it, for the parts closest to the ring will stretch the most. That is, the bending is positive, and the tendency would seem to be not to burst, as Mr. Whitney says, but to collapse downward if the profile should become so far distorted as to result in a reverse curve, concave upward. In that case the extra masonry should be on the outside rather than on the inside, near the bottom, to provide room for the line of stress to move outward without getting too near the surface.

Where the dome has tension in the lower part, the rings taking this tension will stretch about the same as the ring at the springing line, if any, and no dangerous effects on the profile are likely to occur.

At the top, the compression tends to increase the curvature of the profile; on a very flat, thin dome, it might even conceivably cause the formation of a low spot in the middle, like the end of Columbus' egg. The meridional stress would then become tension and the middle would drop out, unless held by meridional reinforcement. All these elastic effects are difficult to compute exactly, especially in masonry with an uncertain elastic coefficient, but the recognition of them is an essential part of the "judgment" that needs to be used in connection with formulas such as these, in which not all the facts are expressed.

Regarding the bending on the faces between the ribs of a pyramoidal dome, Professor Cain's criticism¶ is well taken; the total horizontal force acting on the horizontal beam, L, is not $\frac{B\,L}{D}$, as stated in the derivation of Equation (11).**

Referring to Fig. 11, the horizontal force at the rib is $\frac{B\,L}{D}$, which is composed of the reactions of the two horizontal beams, L, on each side of the rib. Each of these reactions is then:

$$\frac{1}{2}\,\frac{B\,L}{D}\,\operatorname{cosec}\,\phi = \frac{B\,L}{2\,D}\,\frac{D}{d} = \frac{B\,L}{2\,d}$$

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* Discussion of the paper by David C. Coyle, Esq., continued from August, 1924, *Proceedings*.

† Author's closure.

‡ With Gunvald Aus Co., New York, N. Y.

§ Received by the Secretary, August 13, 1924.

‖ *Proceedings*, Am. Soc. C. E., August, 1924, p. 874.

¶ *Loc. cit.*, p. 876.

** *Loc. cit.*, April, 1924, p. 416.

and the total of the forces acting on the beam, L, has $\frac{B\,L}{d}$ as its component perpendicular to the beam.

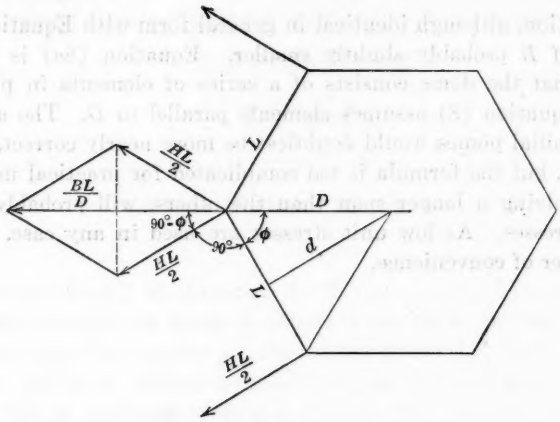

FIG. 11.

As to the distribution of this load there might be assumed a series of sectors parallel to D, which would give a load varying from zero at the center to a maximum at the ends, or a series parallel to d, giving a load varying from zero at the ends to a maximum at the center. It seems reasonable for practical purposes, then, to assume a uniform load and use the coefficient, $\frac{1}{12}$.

The bending would be $\frac{B\,L}{12\,d}$ instead of $\frac{B\,L}{12\,D}$, as previously given, or,

$$-M = \frac{D\,W\,L}{12\,d\,n\,R}\,(1 - \tan^2\theta) - p\,\frac{L^2\,D}{12\,d}\tan\theta$$

rather than Equation (11), as given.

Professor Cain seems to be in error in his derivation of Q, for $P = Q \sin\phi$, instead of $Q = P \sin\phi$ as he states. This error in Mr. Schmitt's paper to which he refers is the reason for the fact, pointed out in the discussion at the time that some of Mr. Schmitt's formulas are incorrect.

The expression for the value of $H\,L$ given by Professor Cain is correct, if R is the radius of curvature of the diametral profile, and α, the angle between R and the vertical. Neither of these quantities can be correlated with the corresponding quantities for the diagonal profile, as their relation depends on the particular form of the profiles. It is possible, however, to derive a value for the ring tension, using Professor Cain's formula. Let

$$H\,L = \frac{W}{n\,S}\sec^2\beta - p\,L\tan\beta$$

in which S corresponds to R, and β to θ, in Equation (8).*

Now,

$$H\,L = \frac{B\,L}{d};\; B = \frac{H\,L\,d}{L}$$

* *Proceedings*, Am. Soc. C. E., April, 1924, p. 415.

therefore, Equation (8*a*) will be :

$$B = \frac{W\,d}{n\,L\,S}\,\sec^2\beta - p\,d\,\tan\beta$$

This equation, although identical in general form with Equation (8), would give values of B probably slightly smaller. Equation (8*a*) is based on the assumption that the dome consists of a series of elements in planes parallel to d, while Equation (8) assumes elements parallel to D. The assumption of elements in radial planes would doubtless be more nearly correct, as Professor Cain remarks, but the formula is too complicated for practical use, and Equation (8), involving a longer span than the others, will probably always give the largest stresses. As low unit stresses are used in any case, the choice is mostly a matter of convenience.

THE ECONOMICS OF HYDRO-ELECTRIC DEVELOPMENT

Discussion*

By Daniel W. Mead, M. Am. Soc. C. E.†

Daniel W. Mead,‡ M. Am. Soc. C. E. (by letter).§—In treating this broad subject, the attempt was made to condense the paper so that it might afford a general but complete outline of the entire question. Being general in character, the outline is subject to modifications when applied to any particular problem, for in such cases certain factors may become insignificant, while others may be accentuated.

The writer limited his analysis in a measure to independent hydro-electric devolopments, although most factors in the outline will apply equally well to a development which is to be a part of a larger power system. As part of a system, the proposed development must necessarily be considered in its relation to the system as a whole, and the aspects of the problem may be considerably changed. Under such conditions the greatest change is perhaps the reduction in the uncertainties as regards the market, the cost of financing, the development of business, the cost of management and operation, and the proportion of possible output that can be utilized.

When a business of considerable size is once developed, its extension is accomplished with much less uncertainty than attends the establishment of a new business. Additions to a profitable business can be financed at much less expense than the development of a new business that is entirely or largely speculative. The cost of management and operation can be based on actual local experience, and the nature of the load and its probable trend of development will be based on conditions fairly well established; thus, the real value of the new plant to the business is more readily and exactly determined and there is less hazard in many phases of the problem. As a part of a system a hydro-electric plant may be developed to a greater machine capacity than would be advisable for a similar independent plant, for it may prove economical, as stated in the discussion, to utilize such a plant for the base load during high water and, if pondage is available, reserve it for carrying peak loads during low water, as under such circumstances capacity can be secured more cheaply than by the installation of additional steam-electric capacity.

* Discussion of the paper by Daniel W. Mead, M. Am. Soc. C. E., continued from August, 1924, *Proceedings*.

† Author's closure.

‡ Prof., Hydr. and San. Eng., Univ. of Wisconsin; Cons. Engr., Madison, Wis.

§ Received by the Secretary, September 4, 1924.

When a hydro plant is a part of a system, other factors besides the load may be subject to similar modification.

The writer has read with interest the discussions of the subject and recognizes that the various illustrations used in the paper* (including Tables 2 and 3 and Figs. 5 and 6, also, Appendix D) are special cases which can be applied only in principle to similar studies. In general, the discussions are elaborations of various phases of the question which might be indefinitely extended, as the subject if treated in detail would require several volumes.

* *Proceedings*, Am. Soc. C. E., April, 1924, Papers and Discussions, pp. 417–452, inclusive.

INCREASING THE CAPACITY OF EXISTING STREETS

Discussion*

By A. C. Janni, M. Am. Soc. C. E.

A. C. Janni,† M. Am. Soc. C. E. (by letter).‡—This timely paper is very apropos of the transportation problem of Lower Manhattan. It is axiomatic that the traffic problem in New York is characteristic of New York; therefore, it should be solved with respect to the peculiar physical shape of the city and with the knowledge of the exigencies of its business life. The general methods of handling traffic in London and Paris, as well as in some of the larger American cities, although useful, as far as the study of their minor details is concerned, cannot be taken as models for other cities where conditions are altogether different.

It is said that in the past few years the traffic in the lower part of New York has increased about 100%, and everything points to a progressive growth year by year. There are two factors in the question which, so to speak, work against each other, namely, the physical conformation of this district and its enormous importance. It is true that part of the congestion is gradually being shifted toward 42d Street, but, thus far, this has not sensibly relieved the condition on account of the constant increase in the population and in the importance of New York with respect to the other markets of the world.

Ways and means for relieving this congestion have been studied and proposed. Some of them are merely makeshifts, not worthy of a metropolis like New York; some are rather expensive, but inadequate; and a few have taken into due consideration the main source of difficulty in this problem.

A map of part of New York (Fig. 16) shows that in Lower Manhattan the north-south transportation is fairly well developed, but the east-west facilities are almost completely neglected; this is one of the causes of the highly congested traffic on the main east and west arteries in this locality at rush hours. The rapid transit lines are congested on the backbone of the island, so that pedestrians are bound to become crowded in reaching their means of transportation. Furthermore, the elevated lines are located on streets which would be excellent north-south arteries of traffic, were it not for the interference of these structures. Finally, for the development of New York toward the north and northeast of Manhattan, adequate long hauling express transportation is lacking.

From 110th Street north the transportation between subways and elevated is well distributed, except for some additional cross connection lines that prob-

* Discussion on the paper by Arthur S. Tuttle, M. Am. Soc. C. E., continued from September, 1924, *Proceedings*.

† Cons. Engr., New York, N. Y.

‡ Received by the Secretary, June 24, 1924.

ably would help it greatly; but this, at least for the present, is not a crucial deficiency. Naturally, in adopting any new transportation plan, the trunks of the elevated lines north of 110th Street should be equipped adequately for their carrying capacity south of 110th Street; that is, the construction of these trunks should be changed to correspond with the new exigencies of transportation.

It is not a precipitate judgment, therefore, to say that the elevated transportation as it is located at present, sooner or later must disappear, to give place to something which fulfills the same task in a more logical and far less obtrusive way. In other words, this type of elevated line must submit to what has already happened in all progressive cities, to the wiring for the telephone, the telegraph, and the trolley car.

It will be seen from Fig. 16 that the new system of transportation proposed by the writer contemplates:

1.—The retention of all present subways, for they are well located and accomplish satisfactorily their work of transportation.

2.—The removal of all present elevated lines from South Ferry to 110th Street.

3.—The substitution of two trunk elevated lines. One should start at South Ferry and run along the Hudson River front to 110th Street, there connecting with the old elevated line (this line, the west line, should have six or eight tracks for expresses and local trains); the other could run from South Ferry to 110th Street along the East River front, as shown, connecting at 110th Street with the old elevated system (this east line could have from six to eight tracks likewise).

4.—A system of cross-connecting subways which would make it possible to transfer to all lines, both elevated and subway, as needed. These changes should be made possible on the basis of a single fare.

It seems desirable that the east and west express lines should have few stations south of 14th Street; and that from 14th Street to 110th Street there should be no stop whatever, so as to make possible the transportation of passengers living north of 110th Street, thus relieving the traffic in Lower Manhattan as soon as possible. It will be seen, also, that the connecting cross subways would suffice to carry people from the center of Lower Manhattan to its margins, and *vice versa*.

Furthermore, passengers, in order to get to the proposed elevated, will either disappear from the surface through the stations of the longitudinal and cross connecting subways, or will walk to the east and west elevated lines. Thus, the great number of pedestrians will be spread over a far larger surface than at present.

The cross subway connections at Fulton, Canal, and 14th Streets will then accommodate practically all the pedestrians, thereby avoiding the great congestion on these streets, which is now intensified in rush hours. For that matter, the entire traffic of Lower Manhattan, as far as pedestrians are concerned, will diminish very materially on the surface of the streets.

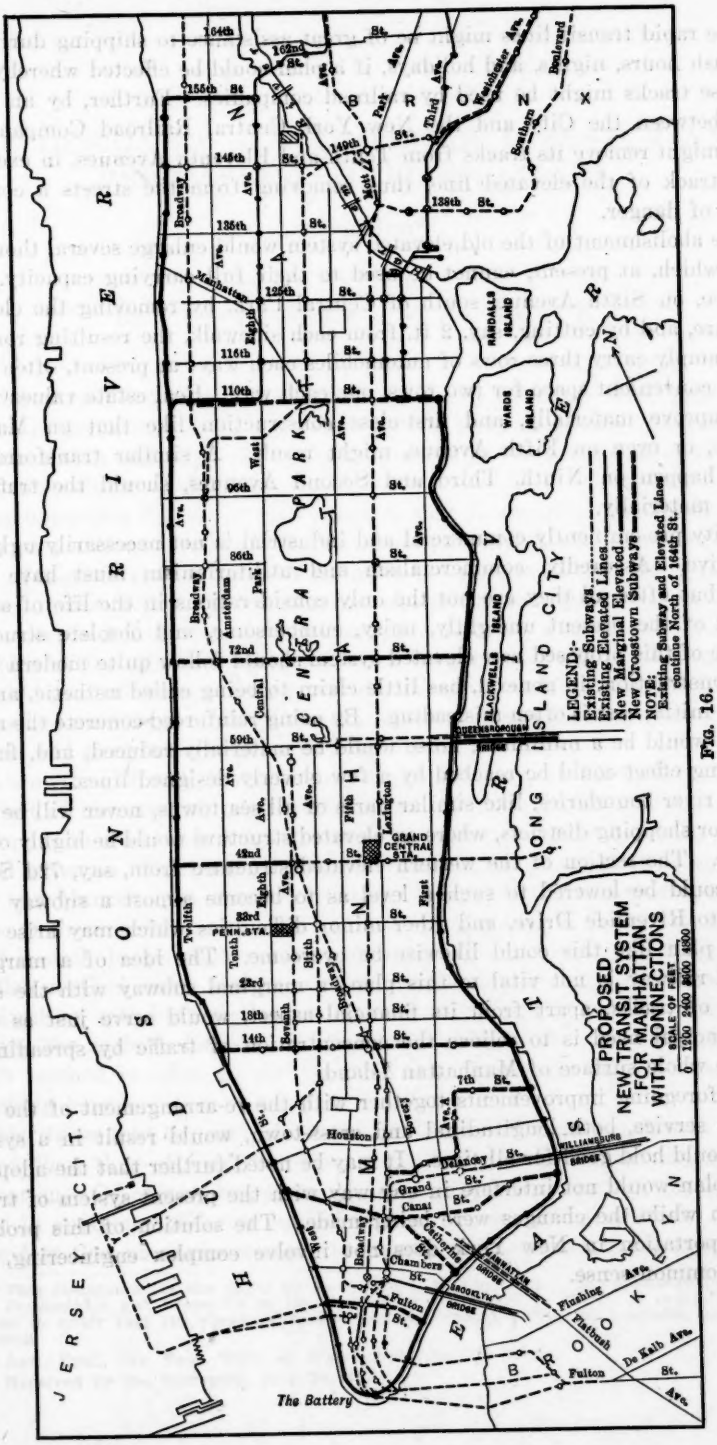
PROPOSED
NEW TRANSIT SYSTEM
FOR MANHATTAN
WITH CONNECTIONS
SCALE OF FEET
0 1200 2400 3600 4800
LEGEND:-
Existing Subways
Existing Elevated Lines
New Marginal Elevated
New Crosstown Subways
NOTE.
Existing Subway and Elevated Lines continue North of 164th St.
HARLEM RIVER
EAST RIVER
NEW YORK BAY
HUDSON RIVER
JERSEY CITY
LONG ISLAND CITY
BROOKLYN
BRONX
MANHATTAN
Broadway
Manhattan
Eighth
Amsterdam
Central
Park
Fifth
Madison
Lexington
First
Second
Third
Park
Boston Road
Webster Ave.
Jerome Ave.
Bedford Park Blvd.
Kingsbridge Road
Fordham Road
164th St.
155th St.
145th St.
135th St.
125th St.
116th St.
110th St.
96th St.
86th St.
72nd St.
59th St.
42nd St.
33rd St.
23rd St.
18th St.
14th St.
Houston
Delancey St.
Grand St.
Canal St.
Chambers St.
Fulton St.
Cortlandt St.
Barclay St.
Vesey St.
Park Place
Murray St.
Warren St.
The Battery
7th St.
Ave. A
Bowery
Williamsburg Bridge
Manhattan Bridge
Brooklyn Bridge
Queensboro Bridge
Blackwells Island
Randalls Island
Wards Island
GRAND CENTRAL STA.
PENN. STA.
Flushing Ave.
Fulton St.
De Kalb Ave.

FIG. 16.

The rapid transit lines might be of great assistance to shipping during the non-rush hours, nights, and holidays, if a plan could be effected whereby some of these tracks might be used by railroad companies. Further, by an agreement between the City and the New York Central Railroad Company, the latter might remove its tracks from Tenth and Eleventh Avenues, in exchange for a track of the elevated line, thus removing from the streets a constant source of danger.

The abolishment of the old elevated system would enlarge several thoroughfares, which, at present, cannot be used to their full carrying capacity. For instance, on Sixth Avenue, south of Central Park, by removing the elevated structure, and by cutting, say, 2 ft. from each sidewalk, the resulting roadway would amply carry three rows of automobiles each way; at present, often there is only convenient space for two rows, one each way. Real estate values would also improve materially, and first-class construction like that on Madison Avenue, or even on Fifth Avenue, might result. A similar transformation might happen on Ninth, Third and Second Avenues, should the traffic be altered materially.

A city pre-eminently commercial and industrial is not necessarily ugly and oppressive. Assuredly, commercialism and utilitarianism must have their places, but after all they are not the only considerations in the life of a city. Instead of the present unsightly, noisy, cumbersome, and obsolete structure, the type of this proposed new elevated system should follow quite modern lines. Steel construction, in general, has little claim to being called æsthetic, and its cheaper initial cost is often misleading. By using reinforced concrete the maintenance would be a minimum, noise would be materially reduced, and, finally, a pleasing effect could be reached by a few cleverly designed lines.

The river boundaries, like similar parts of all sea towns, never will be residential or shopping districts, where an elevated structure would be highly objectionable. The section of the western elevated structure from, say, 72d Street north, could be lowered to such a level as to become almost a subway with respect to Riverside Drive, and other minor difficulties which may arise in a general plan like this could likewise be overcome. The idea of a marginal elevated railway is not vital to this plan; a marginal subway with the same number of tracks, apart from its financial aspect, would serve just as well. The principal need is to relieve the concentration of traffic by spreading it over the whole surface of Manhattan Island.

The foregoing improvements together with the re-arrangement of the surface car service, both longitudinal and cross-town, would result in a system which would hold good for all time. It may be noted further that the adoption of this plan would not interfere in any way with the present system of transportation while the changes were being made. The solution of this problem of transportation in New York does not involve complex engineering, but merely common sense.

THE DESIGN OF A MULTIPLE-ARCH SYSTEM AND PERMISSIBLE SIMPLIFICATIONS

Discussion*

By J. Charles Rathbun, M. Am. Soc. C. E.

J. Charles Rathbun,† M. Am. Soc. C. E. (by letter).‡—This paper is a valuable contribution to the literature on the analysis of the concrete arch ring, containing as it does an Appendix describing the method of analysis by means of the ellipse of elasticity.

One important feature of the paper is that it considers the elasticity of abutments and foundations. Actually, abutments are elastic, likewise foundations. Designers are too prone to compute the stresses of the ring with great accuracy under the assumption that the abutments are rigid. As the central span is designed first with rigid abutments and, later, with elastic abutments some idea of the effect of this change can be gained. Incidentally, the most important part of the structure is the abutment. Nearly all arch failures can be traced to movement of the abutments whereas in the drafting room most of the time and study are given to the ring.

This method of arch analysis is especially adaptable to the investigation of the effect of moving and concentrated loads, but even in the investigation of stresses due to dead loads it is more rapid in application than the more popular method given in "Principles of Reinforced-Concrete Construction", by Turneaure and Maurer. On several occasions, the writer has analyzed arch rings by one of these methods and used the other as a check. A draftsman inexperienced in arch design requires about the same amount of time for the analysis by either method when working under an experienced engineer. However, a man familiar with both methods would save one-third to one-fifth of his time by using the author's method. A large percentage of the time used in Turneaure's method is taken in dividing the arch ring accurately into segments of varying length.

This shows that the author's method requires more study before it can be applied than that of Professor Turneaure, but that it is more rapid in its application. Of course, by means of legitimate approximations the time involved in applying the latter method may be considerably shortened. In view of the fact that an engineer needs to master the theory but once, after which

* This discussion (of the paper by A. C. Janni, M. Am. Soc. C. E., published in August, 1924, *Proceedings* and presented at the meeting of September 3, 1924), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Asst. Prof., Civ. Eng., Univ. of Washington, Seattle, Wash.

‡ Received by the Secretary, July 24, 1924.

he may apply it many times, it is surprising that the graphical method has not proven more popular.

The writer has studied this method with a view to discovering the reason for its lack of popularity. With the exception of poles and polars, the author uses only simple graphical constructions that should be familiar to every structural designer. The author has developed the ellipse of elasticity using the method of Projective Geometry and the Principal of Reciprocity, rather than of Analytic Geometry. This is the more elegant method from a mathematical standpoint, but the curriculum of an American engineering school does not include this branch of mathematics. Although poles, polars, ellipses, conjugate axes, etc., are taught in the undergraduate engineering courses in Analytics, it must be admitted that they are totally forgotten before the student obtains his diploma. The subject could be quickly reviewed, if presented in the language of Analytic Geometry. There is danger that one reading a paper like that of the author assumes that it involves mathematics more advanced than he has acquired or is willing to acquire and, therefore, leaves it unread. Such expressions as "the auxiliary conic of the system is imaginary" are discouraging to any one, even the most enthusiastic theorist.

In explaining this theory, the writer has been in the habit of avoiding any reference to the ellipse as long as possible, bringing it in only after an analytic relationship between the line of force and the center of rotation has been established. To do this two examples are first solved, the first of which is a study of the location of the center of rotation of the end of a simple cantilever beam when deflected by a force applied at the end, but acting along a given line.

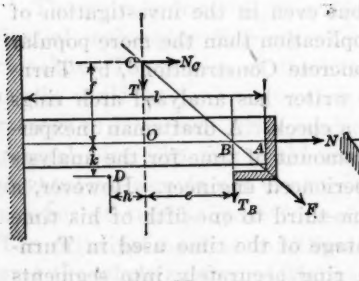

FIG. 22.

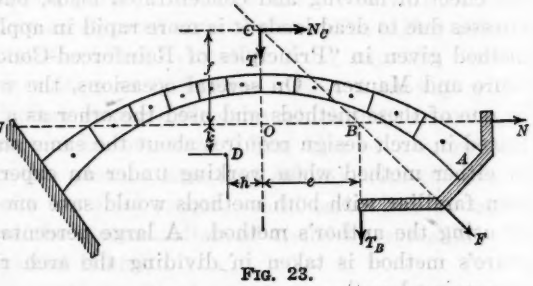

FIG. 23.

Let Fig. 22 represent a horizontal straight cantilever beam the length of which is l, deformed by a force, F, acting along the line, BC, but applied at the point, A. To find the co-ordinates, h and k, of the center of rotation, D, of the point, A, in terms of the equation of the line of force, BC, resolve the force, F, into its vertical and horizontal components, T_B and N, acting through B, the point where F intersects the horizontal axis of the beam (the X—axis). The deflection of A due to T_B acting alone is:

$$\Delta = \frac{T_B l^2}{E I}\left[\frac{l}{12} + \frac{e}{2}\right] + \chi \frac{T_B l}{S A}$$

while the angle of rotation is $\phi = \frac{T_B\, l\, e}{E\, I}$, using the author's notation and the dimensions given in Fig. 22. Both these expressions have been derived in accordance with the common theory of flexure.

If the distance from the deflecting point, A, to the center of rotation is denoted by r (radius of rotation), it follows from Fig. 22, that $r = h + \frac{l}{2}$; but $\Delta = r\,\phi$, whence:

$$h + \frac{l}{2} = \frac{\Delta}{\phi} = \left\{ \frac{T_B\, l^2}{E\, I} \left[\frac{l}{12} + \frac{e}{2} \right] + \chi \frac{T_B\, l}{S\, A} \right\} \div \frac{T_B\, l\, e}{E\, I}$$

Neglecting deformation due to shear, this reduces to $h\, e = \frac{l^2}{12}$. The deflection of the point, A, due to N acting alone, is $\frac{N\, l}{A\, E}$. There is no rotation.

When T_B and N act together, the point, A, rotates through an angle, $\phi = \frac{T_B\, l\, e}{E\, I}$, and at the same time moves vertically a distance, $\frac{l^2}{12\, e}$, and horizontally a distance, $\frac{N\, l}{A\, E}$. This amounts to a rotation through an angle, ϕ, about a center the co-ordinates of which are:

$$h = -\frac{l^2}{12\, e}, \text{ and } k = -\frac{N\, l}{A\, E} \div \frac{T_B\, l}{E\, I} = -\frac{N\, I}{T_B\, A\, e}$$

As h is entirely independent of the values of T_B and N, it follows that the center of rotation of the point, A, due to a force, F, acting through B, lies on the vertical line, $x = -h$. When N is zero, k is zero.

Again, resolving the force, F, into its vertical and horizontal components, T and N_c acting through C, the point where F intersects the vertical through the central point of the beam, the deflection due to T acting alone is obtained by placing $e = 0$ in the foregoing expressions obtained for T_B. $\Delta = \frac{T\, l^3}{12\, E\, I}$, and is vertical, while $\phi = 0$.

The horizontal deflection of the point, A, due to N_C acting alone, is $\Delta = \frac{N_C\, l}{E\, A}$, while $\phi = \frac{N_C\, f\, l}{E\, A}$; whence, $k = \frac{\Delta}{\phi} = \frac{I}{A\, f}$. As k is independent of the values of N_C and T, it follows that the center of rotation of the point, A, due to a force, F, acting through C, lies on the horizontal line, $y = -k$.

It follows that the co-ordinates, h and k of the center of rotation, D, of the point, A, when acted on by a force, F, along the line the equation of which is $\frac{x}{e} + \frac{y}{f} = 1$, are $-\frac{l^2}{12\, e}$ and $-\frac{I}{A\, f}$; and the graphical construction of the author's Paragraph 10* immediately follows.

Except for sign this is the relationship between the pole and polar of a real

* *Proceedings*, Am. Soc. C. E., August, 1924, p. 777.

ellipse as given in standard texts on Analytic Geometry. The change of sign presents no difficulty.

Take as a more general example a curved cantilever beam (Fig. 23) made up of a series of parts the elastic weights of which are G. O is the center of gravity of the system of elastic weights and the X and Y-axes are drawn so that $\Sigma\, G\, x\, y = 0$. If the system is symmetrical these axes are best taken as vertical and horizontal.

The several movements of the point, A, due to the force, T_B, are:

$$\Delta\, y = \Sigma\, T_B\, G\, (x + e)\left(x + \frac{l}{2}\right) = T_B\, \Sigma\, G\, x^2 + T_B\, \frac{e\, l}{2}\, \Sigma\, G$$

$$\Delta\, x = \Sigma\, T_B\, G\, (x + e)\, (y + m) = T_B\, e\, m\, \Sigma\, G$$

$$\Delta\, \phi = \Sigma\, T_B\, G\, (x + e) = T_B\, e\, \Sigma\, G$$

whence,

$$h\, e = \frac{\Sigma\, G\, x^2}{\Sigma\, G} \quad . \quad A\, s' \, \frac{\Delta\, x}{\Delta\, \phi} = m$$

the center of rotation for a vertical force lies on the axis. The other quantities and values may be obtained in a manner similar to those in the first example.

If the arch is unsymmetrical the axes may not be at right angles to each other. Taking advantage of the fact that the center of rotation due to a force parallel to one axis lies on the other, the second axis may be found after assuming the first.

Thus, the author's whole graphical method can be derived algebraically. Although this method is not as finished or as pleasing to those who have specialized in this type of theory as that given in the paper, it has the distinct advantage of presenting to the engineer a minimum of new ideas.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

EDWARD BENJAMIN CUSHING, M. Am. Soc. C. E.*

DIED FEBRUARY 17, 1924.

Edward Benjamin Cushing was born in Houston, Tex., on November 23, 1862. He received his first instruction in the public schools of Houston and because of his rare aptitude for mathematics, advanced with unusual rapidity. He was afterward sent to the Texas Military Institute at Austin, where he remained until the opening of the Agricultural and Mechanical College of Texas, at Bryan, in 1877. He remained at this institution during 1877 and 1878, a member of the first engineering class, intending to complete his course, but his work was interrupted by the death of his father, the late E. H. Cushing, a prominent writer and newspaper owner of Houston. He returned to the college later to resume his studies and, in 1899, received the degree of Civil Engineer.

Mr. Cushing obtained his first practical training in engineering under Col. Ashbel Smith, a noted scholar and at one time Minister Plenipotentiary and Ambassador Extraordinary from the Republic of Texas to the Court of St. James. While engaged with Colonel Smith, Mr. Cushing located the boundaries that marked many a league, including territory covering the shores of Trinity Bay and the Gulf of Mexico, around the San Jacinto Battle-Grounds, and Goose Creek, the richest oil fields in this section of the State. Many of these lines are recognized to-day.

Mr. Cushing then served as an Axeman under the late James P. Converse, Chief Engineer of that branch of the present Southern Pacific Lines, then known as the Galveston, Harrisburg and San Antonio Railway. Later, he was promoted to the position of Chief of Party and located a line from San Antonio, Tex., into Mexico. This work was accompanied by the hard knocks and hazards characteristic of a rough frontier country. In 1881, he was advanced to the position of Engineer in Charge of Construction on the Galveston, Harrisburg and San Antonio Railway.

In 1882, Mr. Cushing was engaged as Assistant Engineer on the location of a line from Piedras Negras to Paila, Mexico, a distance of 200 miles, following which, during construction, he was made Assistant Engineer of Track. In 1883, he accepted a position as Assistant Engineer with the Galveston, Harrisburg and San Antonio Railway Company, and was in charge of re-alignment and grade reduction, as well as yard remodeling. During 1885 and 1886, he served as Assistant Engineer in charge of drafting and accounting for the Gulf, Colorado and Santa Fé Railway Company, during much construction work in Texas and Indian Territory. Later, he was promoted to be Assistant Engineer in Charge of Roadway, Bridges, and Buildings.

* Memoir prepared by H. F. Jonas, M. Am. Soc. C. E.

In 1889, Mr. Cushing returned to the employ of the Galveston, Harrisburg and San Antonio Railway Company, which had been merged with the Southern Pacific Company, as Resident Engineer in Charge of Track and Roadway, which title he retained until 1897. During this time, he was assigned to the rehabilitation of the Houston East and West Texas Railway, extending from Houston, Tex., to Shreveport, La. Later, he was appointed Chief Engineer and General Superintendent of this Company with which he remained for five years.

He was then transferred to the Southern Pacific Lines as Engineer on Maintenance of Way of the Atlantic System, with jurisdiction from New Orleans, La., to El Paso, Tex., including the Houston and Texas Central Railroad—a total mileage of 3 200. In 1916, Mr. Cushing was appointed Assistant General Manager of the Southern Pacific Lines, which position he held until he left for overseas service.

In June, 1917, he received an appointment as Major of the 7th Regiment of Engineers, later the 17th Regiment, which was mustered into service at Atlanta, Ga. The regiment sailed immediately for France, and, after reaching Havre, Major Cushing was transferred to the Railway Transportation Corps. His territory included La Pallice, France, a very important point in the operation of the Expeditionary Forces. On May 25, 1918, he was made Chief Transportation Officer of Base Section No. 6 of the American Expeditionary Forces. In September of the same year, he was promoted to the rank of Lieutenant Colonel in the Transportation Corps, and, in October, he was appointed General Superintendent of the Army Transportation Corps, covering Mediterranean ports. He received distinguished service medals from the Governments of France, Belgium, and Italy, from the General-in-Chief of the American Forces, and the French Legion of Honor.

Colonel Cushing entered the Army with the rank of Major, through test examinations and strong endorsements, although he offered to serve as a Lieutenant rather than miss the opportunity of service abroad. To do so he resigned from an official railroad position that paid him a higher salary than any military position offered.

On his return from France, he resumed his position as Assistant General Manager with the Southern Pacific Lines, Texas and Louisiana, but remained only a short time, having been retired after forty years of service on account of physical disability.

Colonel Cushing was elected to honorary membership in the Engineering Association of the South, and received a similar compliment from the Franklin Institute, Philadelphia, Pa. He was a Thirty-second Degree Mason and a member of the Houston Lodge of the Benevolent Order of Elks.

Among other military honors conferred on Colonel Cushing, were appointments by Governors of two States—Texas and Louisiana—to the rank of Lieutenant-Colonel in crack Guard regiments of those States. In Texas, he was called into active service with the State Guard on more than one occasion, responding each time on short notice and acquitting himself with credit.

He acted as Chairman of the Board of Directors of the Agricultural and Mechanical College of Texas, at a time when the college was temporarily

without a President, and until the present head of the College, Dr. W. B. Bizzell, was appointed. Colonel Cushing also served as President of the Alumni Association.

Soon after his retirement from active railroad service, he was appointed Receiver of the Ranger National Bank, which institution grew to mammoth proportions during the boom days of the Ranger Oil Field, but collapsed under the strain of deflation. Colonel Cushing, held this position at the time of his death in Houston, on February 17, 1924.

He is survived by one brother, two married daughters, four grandchildren, and a host of loyal friends to mourn his untimely death.

Colonel Cushing was elected a Member of the American Society of Civil Engineers on November 1, 1893.

THOMAS REMINGTON HOLDEN DANIELS, M. Am. Soc. C. E.*

DIED NOVEMBER 7, 1923.

Thomas Remington Holden Daniels, the son of George Metcalf and Ellen Eliza (Holden) Daniels, was born in Providence, R. I., on February 19, 1876. His father died when he was nine years of age, so that his education was dependent on his own efforts. He was graduated from high school at the age of sixteen, and then went to work, improving his education by the study of civil engineering at night schools. The greater part of his technical knowledge however was gained by practical experience.

From 1892 to 1896, Mr. Daniels was a student in the office of R. H. Tingley, Consulting Civil Engineer, of Providence, R. I., and from 1897 to 1898, he was with J. A. Latham, Civil Engineer, also of Providence.

In 1898, Mr. Daniels entered the electric railway field as Assistant Engineer in charge of track and roadway of the Rhode Island Company, Providence, R. I. He left that position in 1904 to become Assistant Engineer for Westinghouse, Church, Kerr and Company, on the construction of the new Terminal Station for the Pennsylvania Railroad Company, in New York, N. Y. During his connection with this Company, he was also engaged on preliminary surveys for the high-tension line from Rochester to Syracuse, N. Y. He resigned from the Westinghouse, Church, Kerr and Company in 1906 and took charge as Assistant Engineer of the rebuilding of tracks for the United Railroads of San Francisco after the earthquake. In this capacity he reconstructed some thirty miles of old cable road and also had charge of the installation of special work.

In 1908, Mr. Daniels severed his connection with the United Railroads of San Francisco to become Chief Engineer of Maintenance and Construction for the Terre Haute, Indianapolis and Eastern Traction Company, and as such he assisted in the design and construction of the West Tenth Street Power Plant at Indianapolis, Ind. He left this Company in 1911 to go to Birmingham, Ala., as Chief Engineer of the Birmingham Railway, Light and

* Memoir prepared by T. H. David, M. Am. Soc. C. E.

Power Company. In addition to street railway work, he also had charge of the Power and Gas Division. In 1914, he returned to Indianapolis as Chief Engineer for the Terre Haute, Indianapolis and Eastern Traction Company, in charge of all track work, bridges, and buildings.

In 1911, Mr. Daniels was married to Effie Cade, of Baton Rouge, La., who survives him. He died November 7, 1923, at his Indianapolis residence and was interred at Baton Rouge, La.

He was a man of modest disposition, high ideals, and absolute integrity, and was very conscientious in his work. He was a great lover of good music, being himself a capable pipe organist. He was affiliated with the American Association of Engineers, and was a Scottish Rite and a York Rite Mason, as well as a member of the Shrine.

Mr. Daniels was elected an Associate Member of the American Society of Civil Engineers on February 6, 1907, and a Member on June 19, 1918.

LEWIS SINGER GILLETTE, M. Am. Soc. C. E.*

DIED MARCH 30, 1924.

Lewis Singer Gillette was born at Niles, Mich., on May 9, 1854. He was the descendant of a pioneer New England family founded by Jonathan Gillette who came to America in the *Mary and John* and settled in Dorchester, Mass. Subsequently, members of the family moved to New York, N. Y., and, later, to Niles, Mich.

Mr. Gillette obtained his elementary education in the country school and High School at Niles. In 1872, he went to Minneapolis, Minn., where he entered the University of Minnesota from which he was graduated in 1876 with the degree of Bachelor in Civil Engineering; later, he received the degree of Civil Engineer from the same institution.

After graduation, Mr. Gillette returned to Niles and for two years was engaged in farming. In 1878, he bought the Niles Chilled Plow Works which he operated until 1882, when he returned to Minneapolis to cast his fortunes with that rapidly growing city.

Mr. Gillette was engaged as Right-of-Way Agent for the Hill Railroads in 1882, and, in 1883, he accepted the position of Engineer for the St. Anthony Water Power Company. In 1884, he bought a half interest in the Herzog Iron Works which in 1889 became the Gillette Herzog Manufacturing Company of which he was made President. As head of this institution, he was an important factor in the development of the structural steel business in the West.

In 1899, the American Bridge Company was organized and took over the plant of the Gillette Herzog Manufacturing Company. Mr. Gillette continued for a short time with the American Bridge Company, in charge of all sales west of the Mississippi River, but his growing interests in other lines demanded his

* Memoir prepared by James B. Gilman, M. Am. Soc. C. E.

full attention; he therefore severed relations with that Company in 1900, and from then until his death devoted his time and energy to numerous enterprises in the organization of which he was instrumental. Among these undertakings was the Minneapolis Steel and Machinery Company of which he was one of the founders in 1902. This Company continued the structural steel business in the West so well begun by Mr. Gillette, and he retained his interest in the enterprise, serving on the Board of Directors continuously from its organization until his death. He was also a Director of the Northwestern National Bank and of the Minnesota Loan and Trust Company. He was President of the L. S. Gillette Company and the Plymouth Investment Company, both dealing in Minneapolis real estate, and of the Chippewa Land and Pasture Company.

During the last twenty years of his life, Mr. Gillette was engaged in and devoted much time to matters of public interest. He served as Chairman of the Building Committee of the United States Chamber of Commerce when its home at Washington, D. C., was constructed at a cost of $2 500 000. He was also an Honorary Vice-President of the Chamber and was a Delegate of that Society to International Conferences in Paris, France, and Rome, Italy.

His loyalty to his Alma Mater was evidenced by his action in joining with another alumnus in financing from their personal resources the employment of Mr. Cass Gilbert, of New York, to lay out a comprehensive plan for the enlargement of the campus. This plan has been maintained in the great development that has taken place in the last few years at the University of Minnesota. By his foresight and personal contribution, it is building toward a harmonious and well-designed arrangement instead of erecting a number of buildings with no definite comprehensive plan.

As an active member of Trinity Baptist Church, at Minneapolis, Mr. Gillette took great interest in its work, as well as in the work of other religious institutions in Minnesota. He was a Trustee of Carleton College, Northfield, Minn., and of Pillsbury Academy.

Mr. Gillette was one of the substantial business men of Minneapolis, prominent in all matters advocated for the betterment of the city. His later years were more devoted to business affairs than to engineering, but he was always keenly interested in engineering projects and in the Society. He was an extensive traveler and had a host of friends in all parts of the world. His death occurred at his winter home in Natchez, Miss., and he was buried in Lakewood Cemetery, Minneapolis.

Mr. Gillette was married to Miss Louise E. Perkins, of Minneapolis, in 1878. He is survived by his wife, a son, George L. Gillette, and three daughters, Mrs. Marion Anson and Mrs. Harry C. Piper, of Minneapolis, and Mrs. J. Gilbert Burton, of Mobile, Ala.

He was a member of the Engineers' Club of New York, the University Club of Chicago, Ill., the University and Minnesota Clubs of St. Paul, Minn., and the Lafayette, Minneapolis, and Minikada Clubs of Minneapolis.

Mr. Gillette was elected a Member of the American Society of Civil Engineers on June 7, 1899.

EMIL ROBERT OLBRICH, Assoc. M. Am. Soc. C. E.*

DIED MAY 25, 1924.

Emil Robert Olbrich, the son of Florian and Mary Olbrich, was born on May 22, 1891, in Philadelphia, Pa., where he received his early education. He was graduated from the Philadelphia-Rutledge Grammar School in 1905, and from the North East Manual Training High School, in Philadelphia, in 1908.

Mr. Olbrich was associated with the *Philadelphia Public Ledger* as Advertising Soliciting Agent for a short time, and then entered Pennsylvania State College, from which he was graduated in 1915 with the degree of Bachelor of Science in Civil Engineering.

He then joined the staff of the Bureau of Highways, Philadelphia, where he served for about two years as Highway Engineer-Inspector, on the construction and maintenance of all types of city pavements and urban roads, interspersed with laboratory testing of paving materials.

In April, 1917, Mr. Olbrich accepted a position with the Barrett Company, in charge of a contract with the Pennsylvania State Highway Department for the surface treatment of 1 000 miles of water-bound macadam in Western Pennsylvania. In November of the same year he was appointed Assistant Professor of Civil Engineering, at the Agricultural and Mechanical College of Oklahoma, serving in this capacity until 1920, except for a period of six months in 1918-19 when he obtained leave of absence and was detailed as Instructor in the Naval Officers' Training School, U. S. Navy.

Mr. Olbrich then entered the University of Michigan, and was graduated therefrom in June, 1921, with the degree of Master of Science in Highway Engineering. He served the remainder of that year with the Dow Chemical Company, of Midland, Mich., and in January, 1922, was appointed Construction Engineer with the North Carolina State Highway Commission. He held this position with distinction until October, 1923, when he resigned to serve as Assistant to the Director of the Advisory Board on Highway Research, National Research Council, with headquarters at Washington, D. C., where he remained until his death.

Mr. Olbrich was only thirty-three years of age at the time of his death, and was advancing rapidly toward a distinguished career along the lines of scientific research in highway engineering, for which work he displayed marked aptitude. He rendered particularly valuable service to the North Carolina State Highway Commission, by his assistance in the study and development of the sand asphalt pavement, many miles of which are now being successfully constructed by that Commission. He was widely known among members of the Engineering Profession, to whom the knowledge of his death came as a great shock. His loss is deeply mourned.

He was unmarried, and is survived by his father and mother, and by two brothers and a sister.

Mr. Olbrich was elected an Associate Member of the American Society of Civil Engineers, on November 9, 1920.

* Memoir prepared by Charles M. Upham, Assoc. M. Am. Soc. C. E.